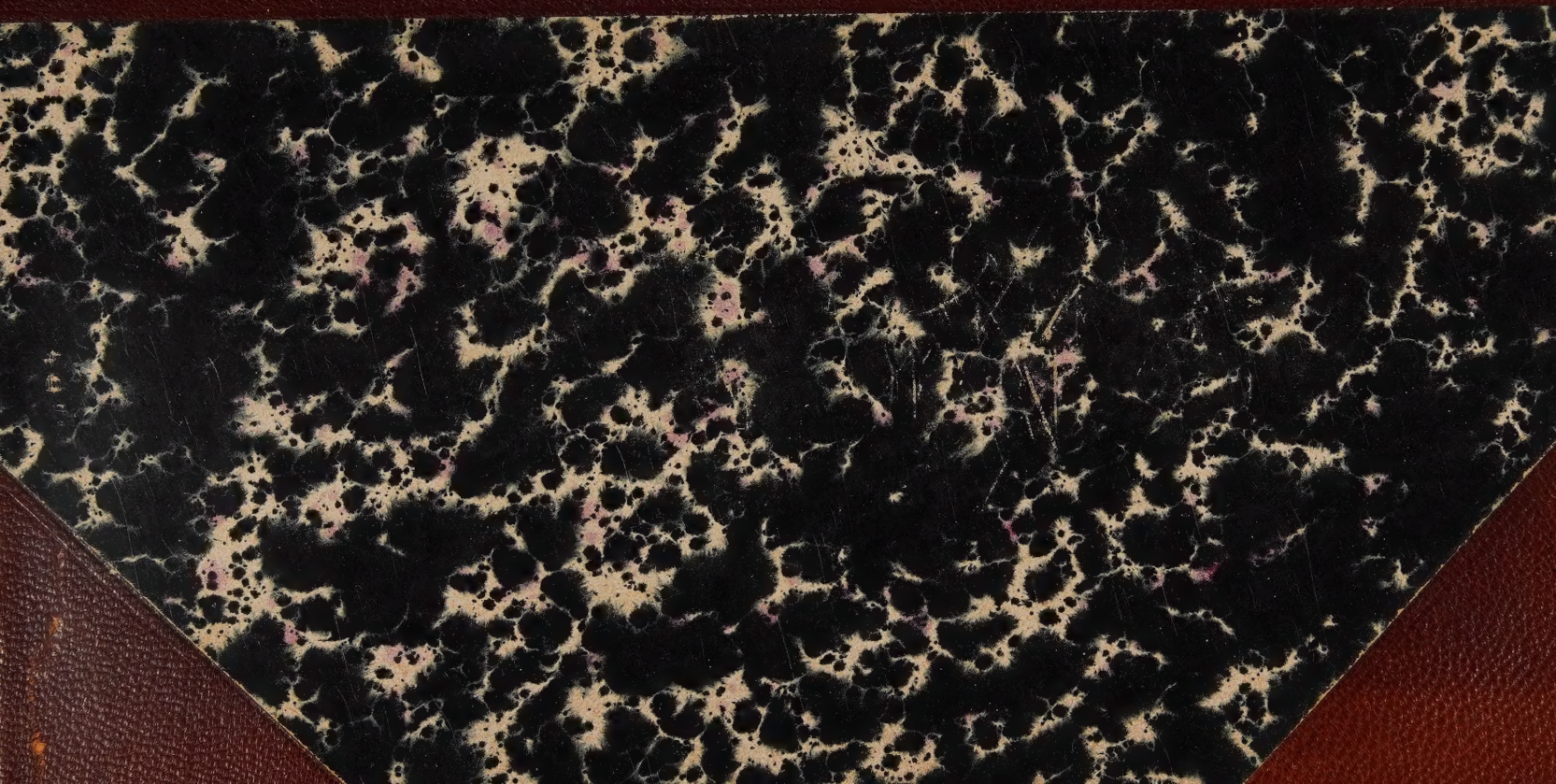


UNIVERSITY OF MAINE
LIBRARY

Class no. ~~620.6~~
Book no. ~~Am3~~ v.73
Accession no. 44,966

TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. LXXIII

SEPTEMBER, 1911

Edited by the Secretary, under the direction of the Committee on Publications.
Reprints from this publication, which is copyrighted, may be made on condition that the full title of Paper, name of Author, and page reference are given.

NEW YORK

PUBLISHED BY THE SOCIETY

1911

839/6

Entered according to Act of Congress, in the year 1911, by the AMERICAN SOCIETY OF CIVIL ENGINEERS, in the Office of the Librarian of Congress, at Washington.

NOTE.—This Society is not responsible for any statement made or opinion expressed in its publications.

CONTENTS

PAPERS

No. PAGE

1194 **ROAD CONSTRUCTION AND MAINTENANCE: AN INFORMAL DISCUSSION:**

(1) Preliminary Investigations.

By Logan Waller Page, W. W. Crosby, Arthur H. Blanchard, Clifford Richardson, C. F. Knowlton, A. F. Armstrong, and Watson G. Clark 2

(2) Relative Value of Three Methods of Carrying on Work.

By Harold Parker, Henry B. Drowne, Joseph W. Hunter, W. W. Crosby, Paul D. Sargent, Fred. E. Ellis, Samuel Whinery, H. B. Pullar, Nelson P. Lewis, and Harold Parker 13

(3) Systems of Maintenance.

By Hubert K. Bishop, George C. Diehl, R. A. Meeker, James H. MacDonald, and Logan Waller Page 25

(4) The Use of Water, Calcium Chloride, Light Oils, Etc., as Dust Palliatives.

By S. Whinery, J. L. Wickes, H. C. Poore, Charles W. Ross, Hubert K. Bishop, L. R. Grabill, R. A. Meeker, and S. Whinery 33

(5) Surface Treatment with Tars, Heavy Oils, Etc.

By Charles W. Ross, W. W. Crosby, Prevost Hubbard, Arthur H. Blanchard, Frank J. Eppele, John R. Rablin, R. K. Compton, William H. Connell, Arthur W. Dean, Fred. E. Ellis, Harold Parker, Michael Driscoll, Philip P. Sharples, Hubert K. Bishop, R. A. Meeker, James Owen, and Charles W. Ross 44

(6) The Use of Bituminous Materials by Penetration Methods.

By W. W. Crosby, Charles W. Ross, R. K. Compton, William H. Connell, Michael Driscoll, Malcolm H. Smith, E. H. Rogers, Alexander F. Armstrong, H. G. Shirley, Paul D. Sargent, R. A. Meeker, James C. Wonders, and W. W. Crosby 74

(7) The Use of Bituminous Materials by Mixing Methods.

By Arthur H. Blanchard, F. C. Pillsbury, Frank J. Eppele, William H. Connell, R. A. Meeker, F. P. Smith, Clifford Richardson, J. W. Howard, Paul D. Sargent, and Arthur H. Blanchard 99

1195 **THE PITTSBURG AND LAKE ERIE RAILROAD CANTILEVER BRIDGE OVER THE OHIO RIVER AT BEAVER, PA.**

By **Albert R. Raymer** 136

Discussion:

By C. W. Hudson 166
Henry S. Prichard 167
Albert B. Hager 169
F. W. Skinner 171
A. W. Carpenter 172
Albert Lucius 173

No.		PAGE
1196	**DAMS ON SAND FOUNDATIONS: SOME PRINCIPLES INVOLVED IN THEIR DESIGN, AND THE LAW GOVERNING THE DEPTH OF PENETRATION REQUIRED FOR SHEET-PILING.**	
	By **Arnold C. Koenig**	175
	Discussion:	
	By W. G. Price	190
	Alexander Potter	191
	T. Kennard Thomson	193
	G. E. P. Smith	193
	Allen Hazen	199
	R. C. Beardsley	203
	Arnold C. Koenig	208
1197	**STREET PAVING CROWNS, WASHINGTON, D. C.**	
	By **T. J. Powell**	225
	Discussion:	
	By E. A. Steece	228
1198	**BOND-FRICTION-RESISTANCE IN REINFORCED CONCRETE.**	
	By **William Fry Scott**	230
	Discussion:	
	By Edward Godfrey	267
	Joseph T. Maguire	271
	A. N. Talbot	274
	Hector Robins Burroughs	276
	Albert I. Frye	278
	F. E. Turneaure	286
	William Fry Scott	289
1199	**THE GOING VALUE OF WATER-WORKS.**	
	By **Leonard Metcalf** and **John W. Alvord**	326
	Discussion:	
	By Clinton S. Burns	356
	Albert I. Frye	361
	Frederic P. Stearns	362
	E. Kuichling	368
	H. F. Dunham	373
	Charles B. Buerger	375
	Halbert P. Gillette	378
	George W. Fuller	384
	Leonard Metcalf and John W. Alvord	386
1200	**ADDRESS AT THE 43d ANNUAL CONVENTION, CHATTANOOGA, TENNESSEE, JUNE 13th, 1911.**	
	By **Mordecai T. Endicott**	392
1201	**SINKING A WET SHAFT.**	
	By **John P. Hogan**	398
	Discussion:	
	By Mason D. Pratt	414
	Charles B. Buerger	415
	L. White	415
	H. M. Hale	417
	H. F. Dunham	418
	T. Kennard Thomson	419

No.		PAGE
	BERTRAND H. WAIT	420
	T. H. WIGGIN	420
	V. H. HEWES	423
	JOHN P. HOGAN	426
1202	**SOME TESTS OF LARGE STEEL COLUMNS.**	
	By James E. Howard	429
	Discussion:	
	By ALBERT LUCIUS	448
	LEON S. MOISSEIFF	449
	A. W. CARPENTER	452
	J. S. BRANNE	457
	LEWIS D. RIGHTS	458
	J. R. WORCESTER	460
	HORACE E. HORTON	461
	ARTHUR N. TALBOT	465
	JAMES CHRISTIE	467
	N. R. MCLURE	467
	R. S. CHEW	469
	GEORGE N. COLE	470
	F. P. SHEARWOOD	471
	CHARLES WORTHINGTON	471
	F. C. KUNZ	484
	JAMES E. HOWARD	489

MEMOIRS OF DECEASED MEMBERS

	PAGE
LEFFERT LEFFERTS BUCK, M. AM. SOC. C. E.	493
GEORGE BOWERS CALDWELL, M. AM. SOC. C. E.	498
FRANK LESLIE DAVIS, M. AM. SOC. C. E.	500
JAMES PALMER FRIZELL, M. AM. SOC. C. E.	501
JOHN FRANKLIN HINCKLEY, M. AM. SOC. C. E.	504
HENRY RANDOLPH HOLBROOK, M. AM. SOC. C. E.	506
LOUIS EDWIN HAWES, ASSOC. M. AM. SOC. C. E.	508
ALBERT VICTOR KELLOGG, ASSOC. M. AM. SOC. C. E.	510

PLATES

PLATE		PAPER	PAGE
I.	The Pittsburg and Lake Erie Railroad Cantilever Bridge Over the Ohio River at Beaver, Pa.	1195	137
II.	Main Tower Posts, Bridge at Beaver, Pa.	1195	139
III.	Anchorage Details; and Cast-Steel Bearings for Tower Posts, Bridge at Beaver, Pa.	1195	141
IV.	Castings Supporting Tower Posts; and L_{10} "Stub" in Process of Manufacture.	1195	143
V.	Putting in Place the Pin Supporting Diagonal Member, Bridge at Beaver, Pa.	1195	145

PLATE		PAPER	PAGE
VI.	Method of Transporting "Stubs"; and Shop Assembling	1195	147
VII.	Main Tower Posts During Erection, Bridge at Beaver, Pa.	1195	149
VIII.	Table 1.—Record of Full-Sized Eye-Bar Tests, Ohio River Bridge, Pittsburg and Lake Erie Railroad, 1909...	1195	151
IX.	Plan and Cross-Section of Floor, Ohio River Cantilever Bridge	1195	153
X.	Erection of Traveler and Anchor Arm, Ohio River Cantilever Bridge	1195	155
XI.	Erection of Anchor Arm; and View of Creeper Traveler Erecting Cantilever Arm	1195	157
XII.	Erection of Cantilever Arm, Ohio River Cantilever Bridge	1195	161
XIII.	Erecting Fixed Span with Traveler; and Bridge Material Delivered on Barges, Ohio River Cantilever Bridge	1195	163
XIV.	Closure of Suspended Span, Bridge at Beaver, Pa.	1195	165
XV.	Diagram Showing Ultimate Bond-Friction-Resistance on Plain Rods and Bars	1198	253
XVI to XXVIII.	Tests of Bond-Friction-Resistance in Reinforced Concrete Beams	1198	303 to 325
XXIX.	Profile in Vicinity of Shaft No. 4, Rondout Siphon	1201	401
XXX.	Pump Chamber in the Side of Shaft No. 4, Rondout Siphon	1201	409
XXXI.	Water-Measuring Device, Shaft No. 4; and Air-Lift	1201	413
XXXII to XXXVI.	Compression Tests on Large Steel Columns	1202	431 to 439
XXXVII.	Compressive Strains, Large Steel Columns	1202	441
XXXVIII.	Strain Measurements, etc., on Large Steel Columns	1202	443
XXXIX.	Strain Measurements and Permanent Sets, Large Steel Columns	1202	443
XL.	Permanent Sets, etc., Large Steel Columns	1202	445
XLI.	Large Steel Columns after Tests	1202	445
XLII.	Large Steel Column after Testing; and Column in Testing Machine	1202	447
XLIII.	Results of Tests of Pin-Ended Wrought-Iron Columns	1202	455

ERRATA.

Transactions, Vol. LXXIII.

Page 92, line 25: For "H. S. Shirley, Esq." read "H. G. Shirley, Esq."
Page 195, line 25: For "overflow" read "underflow."

Mr. G. E. P. Smith has also requested that the following changes be made in his discussion on Mr. Koenig's paper:

Page 196, lines 6 and 7 from bottom: Strike out "$Q = C\frac{H\,S}{L}$, in which S is the cross-sectional area," and substitute "$Q = C\frac{H\,S}{l}$, in which S is the cross-sectional area, and l is the length of element."

Page 197, lines 13 and 14: Strike out the sentence beginning "The gradient is subject" and substitute "The gradient is measured upward from the base of the dam, and is dependent, therefore, on the depth of the base below the river-bed. In cases of standing back-water, or of discharge over the dam, the necessary changes in the position of the gradient are obvious."

Page 197, line 18: Between the words "dam" and "is" insert "as is done by the empirical formula used by engineers in India."

Page 198, line 7 from bottom: For "a sine" read "an anti-sine."

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1194

ROAD CONSTRUCTION AND MAINTENANCE.

An Informal Discussion Presented at the Meetings of January 20th and 21st, 1911.

PAGE

(1) "Preliminary Investigations" 2

(2) "Relative Value of Three Methods of Carrying on Work":
(*a*) That in which both labor and material are furnished by the contractor.
(*b*) That in which the material is supplied by the party of the first part, and the labor by the contractor.
(*c*) That in which both the labor and material are supplied by the party of the first part 13

(3) "Systems of Maintenance" 25

(4) "The Use of Water, Calcium Chloride, Light Oils, etc., as Dust Palliatives" 33

(5) "Surface Treatment with Tars, Heavy Oils, etc." 44

(6) "The Use of Bituminous Materials by Penetration Methods." .. 74

(7) "The Use of Bituminous Materials by Mixing Methods." 99

(1) PRELIMINARY INVESTIGATIONS.

By Messrs. Logan Waller Page, W. W. Crosby, Arthur H. Blanchard, Clifford Richardson, C. F. Knowlton, A. F. Armstrong, and Watson G. Clark.

Mr. Page.

Logan Waller Page, M. Am. Soc. C. E.—There are at the present time many features in American practice which render the satisfactory application of preliminary investigations to highway construction unnecessarily difficult. It is the object of this discussion to point out some of the most prominent of these obstacles and to suggest certain means of remedying them.

The present status of experimental road work in the United States is far from satisfactory. It must be admitted, however, that much progress along experimental lines has been made during the past few years by certain individual investigators and public service bodies, and a general improvement in methods of construction and plans for the investigation of previous work is noticeable throughout the country. The best that can be said, however, is that a start only has been made along the right lines, and that much yet remains to be done.

Since 1907 a great number and variety of experiments with dust palliatives and road binders, in the treatment and construction of roads, have been conducted by American road engineers, but the greater part of this work has been of practical value to the experimenter only. Many valuable experiments have not been published in such form as to assist others engaged in similar work, and the general criticism may be made that in the majority of published reports the data presented are both meager and incomplete. Moreover, the results obtained from these experiments have not been properly correlated or digested. This, of course, may be accounted for, to some extent, by the fact that sufficient time has not yet elapsed to obtain final results, particularly in experimental road construction.

Without entering into detail, it may be said that at present there is a lamentable lack of knowledge among road engineers, as a class, regarding even the types of dust preventives and road binders now obtainable; and in many cases there is complete ignorance concerning the physical and chemical properties of such materials. This seems to be a rather unfortunate state of affairs when it is realized that such knowledge is essential to their intelligent use.

Without in any way belittling the good work already done, and the earnest endeavors of many engineers, one other criticism may be made in regard to the experimental road work of the past. This refers to the lack of co-operation among experimenters regarding the general lines which have been followed. Thus, in certain instances one may find that a bituminous material used satisfactorily by one experimenter in construction work has been used by another in surface treatment with

bad results, as might have been expected. Without attempting to ascertain the cause of failure, one will discredit this particular material while the other will extol it. A third party wishes to surface-treat or construct a road, and, hearing opposite opinions from the first two experimenters, is at a loss to know whether or not to use the material in question. As a rule, without regard to the experience of either, or taking the trouble to find out the cause of the difference of opinion, he decides to make an experiment of his own, and often repeats mistakes which have previously been made. Mr. Page.

To remedy this condition of affairs, the following suggestions are offered:

(1) That all engineers interested in modern road problems make a special study of bituminous and other road materials, with regard to their origin, classification, methods of production or manufacture, and their physical and chemical characteristics;

(2) That a systematic examination be made of all dust preventives and road binders used in experimental work, and that complete reports of such examination be included in all published descriptions of experiments;

(3) That a detailed description of the method of application or construction be made in all published reports, and that a statement of all peculiar local conditions in connection with each experiment be included in these reports;

(4) That, whenever practicable, a careful traffic census be taken for all roads which have been surface-treated or constructed with bituminous or other binders;

(5) That more attention be paid to comparative experiments carried on under a given set of conditions, where various road binders are used according to one or more methods;

(6) That experiments be conducted with the view of determining the relative economic value of surface treatment and construction with various road binders, especially in connection with the traffic to which the road is subjected.

The first of these recommendations may be passed without further comment. With regard to the second, it may be said that the methods of examination of road binders should be standardized, in order that the results of examination be properly interpreted by all. For a number of reasons, such standardization is a difficult matter to accomplish, but it is to be hoped that certain standards will be generally adopted in the near future, as a result of the work of this Society and the American Society for Testing Materials.

With regard to the construction of experimental bituminous roads, it is suggested that more attention be paid to the economical development of the mixing method, which, in the speaker's opinion, will eventually prove best adapted for the more important highways.

Mr. Page. In the speaker's judgment, the importance of taking a traffic census has been greatly underestimated in the United States. It is quite evident that the selection of a suitable type of road for any locality should be governed largely by the volume and character of traffic to which the road will be subjected, and yet but little attention is commonly given to this matter. Attention is rarely given to more than the general nature of the traffic; that is, whether it is heavy or light, and whether it consists mainly of motor vehicles or horse-drawn vehicles. What is considered as heavy traffic for one locality may be considered as light for another, and, therefore, it would seem to be most important that a detailed record be made of both the character and the volume of traffic for each individual road of importance.

With regard to comparative experiments made under the same general conditions, there is one point which should be emphasized, namely, that the results of such experiments should be interpreted, not necessarily as showing unquestioned comparative values, but mainly comparative values under a given set of conditions. This leads to a closely related subject which may be mentioned in conclusion, that is, the futility of attempting to formulate standard specifications for bituminous road materials. It has been the speaker's experience that a given binder, used according to a prescribed method, with satisfactory results under a given set of conditions, may prove quite unsatisfactory when used according to the same method, under a different set of conditions. The framing of satisfactory specifications for bituminous binders, therefore, must continue to be, as in the past, a matter of expert judgment.

Mr. Crosby. W. W. Crosby, M. Am. Soc. C. E.—In the work of preliminary investigation it is necessary to consider how the actual performance of the work to be accomplished may be carried out, before a decision as to the method to be adopted may be reached.

Owing to the fact that the speaker has so frequently heard expressed the desire for a panacea or cure-all for road-ills, in the way of a particular method of using any material, and that he feels there is still remaining, in some quarters at least, a mistaken idea of his attitude on these questions, in spite of his previous utterances on this subject, he desires to make a brief statement of his position.

Any tendency to establish a panacea for either method or material is to be regretted, because any such idea is improperly based and impossible of accomplishment, and because each problem should be solved individually.

The speaker believes that even the relative values of the four common methods in use, namely, Palliatives, the Surface Treatment, the Penetration Method, and the Mixing Method, have not been as clearly established from observed and recorded facts as is necessary for a final opinion as to which method should be used, even when all

other circumstances in the case may be accurately known. The speaker considers that each method has its limitations, though they may not be sharply defined at present, and that there are many good arguments for each. Mr. Crosby.

Palliatives have their main use on roads of light traffic, already in good condition as to surfacing; where excessive smoothness is undesirable; where there are esthetic objections to other methods; where their convenience of application demands serious consideration; and where annual cost may not be a controlling factor.

Surface treatments are limited to roads with good surfaces; of moderate traffic; where a fair degree of smoothness of surface and the resulting appearances from "glaze" or color may not be objectionable; where the cost under the traffic may not be excessive; and where the convenience of their application is desirable of consideration. It will probably be generally acknowledged that the records as yet are not clear as to the amount of traffic that can be cared for successfully by the surface treatment, or as to its economy under known traffic conditions.

Penetration methods are limited to the resurfacing of old roads or the construction of new ones. Their relative economy is by no means clear as yet, nor is their relative efficiency under known traffic conditions apparent. A great argument in favor of their adoption—in addition to the theoretical ones which have probably been covered too fully in the past to need repetition here—has been the practical one that it was so much easier to induce lay boards, or officials, controlling the work, to "sprinkle some pitch on the stone before applying the chips," than it was to induce them to invest a portion of their limited funds in the equipment necessary for the mixing method. Undoubtedly, this condition of affairs has also been furthered by the attitude of certain patentees, who have so often made such broad claims, as to the extent of their patents, as to scare many laymen into avoiding any mixing method, even when they were convinced that some such might be desirable, and were ready to make the investment for equipment. Consequently, the penetration method has secured an additional argument, perhaps in some cases unjustly, though the speaker wishes to avoid any lack of clearness as to his opinion. He still firmly believes in his previous arguments as to the correctness of the principles underlying the method, and he submits that as yet no appreciable amount of direct evidence has been recorded to disprove either these theories or the actual value in practice of the method for many cases. The penetration method is probably limited in its actual use to roads with more than a minimum of traffic, but what that minimum is, as yet, is not satisfactorily clear; to roads where extreme firmness and, perhaps, smoothness of surface are not objectionable; where the dark color is not a serious cause for criticism; and where the cost in the

Mr. Crosby.

long run is not excessive. It may be an especially valuable process for reconstruction.

The mixing method is also limited in application to reconstruction or new construction. It may be said that the first three methods apply to maintenance and the last two to construction, making the third (the penetration method) lap the dividing line. The mixing method is also probably limited in use by the same considerations as in the case of the penetration method, though the difference in first cost of the two methods may result in some variation in the solution of a problem.

DIAGRAMMATIC ARRANGEMENT OF CONTROLLING FACTORS INFLUENCING THE SELECTION OF THE METHOD TO BE ADOPTED.

Method to be used:

- Physical Suitability of Results:
  - Esthetics;
  - Traffic;
  - Smoothness, or Firmness;
  - Dustiness, or Cleanliness;
  - etc.
- First Cost:
  - Equipment;
  - Cost of materials;
  - Cost of skilled labor;
  - Cost of ordinary labor;
  - Workable season;
  - etc.
- Cost of Maintenance:
  - Repair equipment;
  - Cost of materials;
  - Cost of using materials;
  - Workable season, and interference with repairs;
  - Frequency of repairing;
  - etc.
- Human, or Personal Elements:
  - Control by laymen, or by engineers;
  - Local or personal prejudices or desires;
  - Knowledge of facts by controlling authority;
  - Tendency of controlling authority to accept other facts or theories;
  - etc.

Further divisions of this diagram may be carried out almost indefinitely. That which is shown will serve for the illustration intended.

The value of a definite form of treatment is largely dependent on the amount and character of the traffic. The present method of taking

records is to count the traffic under certain classifications, as one-horse, two-horse, three-horse, etc., for the horse-drawn traffic, and the light and heavy motor cars and motor trucks for the motor-car traffic. In the speaker's experience, a comparison of records taken in this manner is extremely difficult. For instance, on various roads, he has records which have been accumulating for a number of years, and, even when they are all tabulated, it is difficult to compare the results. Mr. Crosby.

If engineers will experiment a little in obtaining such information, and assign a unit to each class of traffic, so that the data can be reduced to units, the comparison of results on different roads will be simple. As an illustration, ridden horses might be given a value of 1, single-horse teams the value 2, two-horse teams the value 4, motor cycles the value 1, runabouts or motor cars in general, except trucks, the value of even double the number of seats in the car, and motor trucks a value of 1 for each 250 lb. of capacity; then the traffic could be reduced to a single unit for purposes of comparison.

Such a scheme is suggested with the idea of obtaining a method by which results can be compared, and so that engineers can judge of the relative value of the different treatments, methods of construction, and possibly of the different materials used.

To sum up: The selection of a method to be used for a particular case, when arranged diagrammatically, may look as complicated as the pedigree of an Arabian horse traced back even a few generations.

If the problem seems formidable at present, it should not be forgotten that it is surely capable of simplification. It is the duty of each engineer, who is interested in the development of the science of the Profession, to exert himself courageously for the elimination of certain improper factors, and for the substitution of facts for the unknown factors at present perhaps algebraically represented, as carefully as recorded experience and co-operation will enable him to dictate, so that the "answer" may be sharp and clear.

ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—The primary object of preliminary investigations is to enable an engineer to determine within certain limits the method of construction or maintenance and the kind of road material which are adaptable, from the standpoints of economics and efficiency, to local conditions. Mr. Blanchard.

It is hoped that by the publication and dissemination of complete data relative to the construction and maintenance of roads, and by the co-operation of engineers through the medium of Committees appointed by National Societies of Civil Engineers, the limits of adaptability of methods and materials may be defined more clearly each succeeding year.

Preliminary investigations should include a consideration, and, in many cases, a thorough investigation, of many of the variable influential factors mentioned in the following discussion.

Mr. Blanchard.

In the study of the traffic to which the road is subjected, it is essential to ascertain the amount and the character of the different types of horse-drawn vehicle traffic and motor-car traffic during the various seasons of the year; the normal and abnormal speed of various classes of traffic; the direction of traffic; the parts of the road occupied by various kinds of traffic; the nature of horses' shoes and non-skidding devices used; the traffic regulations in force; and the probable change in the character and amount of the traffic, due in part to the improvement of the road in question or of other roads.

Engineers familiar with the problems of modern highway construction will recognize the influence of the foregoing elements on the ultimate design. For example, a traffic census showing intense and heavy commercial traffic evidently calls for a different type of foundation than the combined traffic of the same number of vehicles but of the passenger type. Again: from the standpoint of the preservation of the surface, a traffic of 200 motor cars per day may or may not necessitate the use of a bituminous material in the construction or maintenance of a road, the combined speed and weight of the motor-car traffic being the controlling factor.

Physical local environments and conditions affect the design materially. The following are among those which should be noted: topographical and geological structure and features; condition and character of cross-roads; effect of the character of the surface of the proposed road on the locality, from the standpoint of esthetics; character of existing surface, foundation, and sub-soil; relation of road to system of roads, from the standpoint of possible diversion of traffic during period of construction; existing and proposed grades and crowns; months available for construction and maintenance work; climatic conditions, covering range of temperature and amount and distribution of rainfall and snowfall.

The availability of the various materials which might be used in construction and maintenance has not, in many cases, received the requisite amount of attention. The investigations along this line should cover a thorough study of the cost, delivered on the site, and the physical and mineralogical character, of all rocks, sands, gravels, etc., which might be used on the road. The source of supply of palliatives and binders should likewise receive attention; and the effect of specifications covering the various physical and chemical properties on the ultimate cost of construction or maintenance is worthy of consideration. In some cases, the method of delivery of bituminous materials has affected the cost and the use of certain methods; hence the location of the road, relative to railroads or storage plants, should be ascertained. The character of available labor, the wages paid, and the method under which the work can be accomplished, may also affect the design.

Mr. Blanchard.

The subject of plant equipment for the construction and maintenance of bituminous surfaces and bituminous pavements has not yet received the consideration warranted by the importance of the subject. It is obvious that, as a result of thorough preliminary investigations, it will be possible, within reasonable limits, to adopt various types of construction and maintenance for a system of highways, in which case it is likewise self-evident that the acquirement of plant equipment will reduce materially the cost of both construction and maintenance under certain conditions.

Mr. Richardson.

CLIFFORD RICHARDSON, M. AM. SOC. C. E. (by letter).—The writer is not a great believer in preliminary investigations at the present time, for several reasons, and is inclined to think that they will result in confusion. The problem of the construction of roads which will meet the conditions imposed by modern motor traffic should be dealt with cautiously. In the United States, at least, attempts to solve it have extended over such a short period of time, that as yet no data are available which offer a basis for investigation. The situation is still purely experimental. What engineers are looking for is success. When this is attained, the manner in which it has been reached can be investigated.

While theories may be of some little value, it is only from experience and service tests that definite conclusions can be safely drawn. Analogies, however, may aid in directing our efforts. Highway engineers, up to the present time, have failed to benefit to any considerable extent from the experience gained in laying bituminous pavements.

It does not seem to be wise at this early stage of the development of bituminous highway construction to draw any conclusions, except those based on analogies, or to make any recommendations, the final results of which can be imperfectly foreseen, for, undoubtedly, they will have to be changed or abandoned in the future. A new industry may be restrained seriously in its development by attempting to prescribe methods of construction which are not based on successful results as demonstrated by service tests. To avoid falling into error, one must await such results and then determine the manner in which these successful results have been obtained, in order to be able to duplicate them. Efforts in any other direction will be in vain. As a parallel, attention may be called to the long period of time which elapsed before it was possible to place the sheet-asphalt paving industry on a rational basis. This was finally accomplished after there were available for study a number of asphalt surfaces which had given satisfactory results for 10 to 15 years or more, under traffic of various kinds. As yet, no bituminous highway surfaces are of sufficient age to furnish the necessary data for a similar study, except in the case of bituminous concrete. The prospect of eventual success of bituminous construction on highways, however, is highly encouraging;

Mr. Richardson. that is, there seems to be little question but that some of the present forms of construction, or modifications of them, where tried materials are used, will prove to be a vast improvement over the water-bound broken-stone road, both from the point of view of resistance to the destructive effect of motor travel, and of economy and unit cost per ton-mile of haul.

The data, in connection with any form of construction, which should be accumulated at the present time are: accurate records of first cost; of the cost of maintenance during the life of the surface; the traffic which it carries, as revealed by traffic censuses at intervals, and the environment under which this traffic is carried, in order that the actual cost of carrying it for the life of the surface may be accurately determined. With such data available, the economy of any form of construction may be compared with data of the same description for any other form. It does not seem possible that any definite conclusions of value can be arrived at, at present, or until such data as have been mentioned are assembled. When a successful form of bituminous construction has been demonstrated, investigation will be of value to determine how this can be repeated, but, until this has been achieved, the writer believes that investigations will be of little value.

Mr. Knowlton. C. F. Knowlton, Esq.—In preliminary investigations, the amount of shade on a road should be given some attention. In determining the method by which a road should be constructed, it has been the speaker's practice to consider whether or not the road was shaded.

Mr. Armstrong. A. F. Armstrong, M. Am. Soc. C. E.—The New York State Highway Department has a special form for traffic census reports, which permits of noting conveniently and rapidly the different kinds of traffic using a road, and of comparing in a concise and direct manner the total amount of traffic on different roads, or on the same road at different times.

When traffic on different roads is compared by counting the vehicles of each kind passing over each road in a stated period, the means of comparison is lengthy and unwieldy, and does not permit of comprehending rapidly the relative amount of traffic.

In the left-hand column of the form used are named: First, the different kinds or units of horse-drawn traffic, motor traffic, and miscellaneous traffic, such as traction engines, etc. The space in the center of the form, opposite each traffic unit, is used for keeping a tally of the vehicles of that traffic unit. The total of this tally for each traffic unit is written in a column near the right side of the form, headed "Direct Total," and the weighted total of each traffic unit, obtained by multiplying the direct total by its proper relative weight, is entered in the column at the extreme right of the form, headed

Mr. Armstrong.

"Weighted Total." Horse-drawn and motor traffic units, according to the arrangement of the form, are kept separate, as are the totals of each of these classes. Summation of the direct totals of the different traffic units of each of these classes gives the total number of horse-drawn vehicles and motor vehicles observed, and summation of the weighted totals gives the total weighted value for each class of traffic. The relative weights used are:

	Relative weight.
For Horse-Drawn Traffic:	
Horse, without vehicle	1
One-horse vehicle, light	2
One-horse vehicle, heavy	3
Two-horse vehicle, light	3
Two-horse vehicle, heavy	4
Three-horse vehicle, heavy	5
Four-horse vehicle, heavy	6
For Motor Traffic:	
Motor cycle	1
Two-passenger car	2
Three-passenger car	3
Four-passenger car	4
Five-passenger car	5
Six-passenger car	6
Seven-passenger car	7
Freight trucks, omnibuses, etc	10
For Miscellaneous Traffic:	
Traction engine	15
Two-traction engine	30
Miscellaneous heavy traffic	5 and upward.

A different set of relative weights might be used, but it is believed that these simple weights give sufficiently accurate results.

The speed, weight, and kind and width of tires, of each traffic unit are not considered, as this would complicate the tally sheet, bring in uncertainties as to the observer's estimate of speed, weight, and width of tires, and be impracticable when observations are taken by one observer of the grade of patrolman or laborer. Furthermore, if the roads observed have somewhat similar grades, surfaces, and alignments, as is the case on most of the improved New York State Highways, these conditions may be assumed to compensate each other, more or less, without introducing a very considerable error in the result.

The report of the New York State Highway Department for 1909 shows the results of the traffic census taken at four different periods, on approximately 500 different roads, the figures given in each case being the total weighted value for both horse-drawn and motor traffic, which permits of determining at first glance the relative amounts of traffic on the various roads at different times stated.

Mr. Armstrong. To be strictly comparable, traffic census reports should be taken over similar periods of time, and, if desired, the results obtained may be reduced to an hourly basis.

Mr. Clark. Watson G. Clark, M. Am. Soc. C. E.—The speaker thinks that too much care cannot be taken in seeing that bituminous binders come up to the specifications. In former years engineers were obliged to test Portland cement in order to ascertain whether it was up to the standard before allowing it to be used. At present, however, under ordinary conditions, if Portland cement is specified, it is pretty certain, without making tests, that it is all right.

If contractors are to be held responsible for the construction and maintenance of bituminous-bound macadam roads, and be blamed for what possibly may be the faults of the companies furnishing the bituminous materials, it is going to be expensive and difficult to obtain the desired results.

In Massachusetts and several other States particular pains are taken to see that the oils purchased are up to the standard of the specifications. The Massachusetts Highway Commission takes a sample from every tank car when it leaves the works, and it is examined and tested before any of the material is allowed to be used on the road. This may meet conditions where tank cars can be used, but in many cases the work is in such locations that the bituminous material has to be shipped in barrels. When a large shipment of bituminous binder is made in barrels, the speaker, so far, has been unable to find any means of assuring himself that all those barrels have been taken from the same mix, and that the results obtained with the latter part of the invoice will be the same as with the first part.

Engineers who specify bituminous binders should have physical and chemical tests of such materials, and the specifications should require that the contractor furnish, with each shipment of the material, a guaranteed chemical analysis by the firm manufacturing it. The analysis should be made from samples taken at the factory at the time of filling the barrels or tanks, and the markings on each barrel or tank should conform with the number of the batch, as indicated on the analysis certificate.

(2) RELATIVE VALUE OF THREE METHODS OF CARRYING ON WORK:

(*a*) That in which both labor and material are furnished by the contractor.

(*b*) That in which the material is supplied by the party of the first part, and the labor by the contractor.

(*c*) That in which both the labor and material are supplied by the party of the first part.

By Messrs. Harold Parker, Henry B. Drowne, Joseph W. Hunter, W. W. Crosby, Paul D. Sargent, Fred. E. Ellis, Samuel Whinery, H. B. Pullar, and Nelson P. Lewis.

Harold Parker, M. Am. Soc. C. E.—In opening this discussion, the speaker will state the reasons why, under the conditions which he will mention, the affirmative position should be taken on each of the three divisions under discussion. Mr. Parker.

It must be clear, to any engineer in charge of construction of any sort, that the various conditions under which the work is to be done should control his action in asking for proposals from contractors, and, depending on such conditions, one of the three methods would from necessity have to be adopted.

If it is desired that the cost of the work shall not exceed a fixed sum, and that the estimate previously made by the engineer is to be corroborated, the method defined under *"a"* would be preferable. This would occur in many instances where public work is involved and the cost of its execution is limited by estimates previously prepared. In such cases there does not appear to be any question as to the wisdom of having the contractor furnish both the labor and materials.

In the practice of all constructing engineers, the second method, that is, "materials supplied by the party of the first part, and labor by the contractor," must frequently be found to be the best, if not the only one, by which the work can be done economically and satisfactorily. It has the advantage that the engineer, desiring to use in the work certain types or brands of materials, the cost of which he knows, can be perfectly sure of the final cost before the work is begun. In this case all contractors who bid are placed on complete equality, and, therefore, as an intelligent and comprehensive offer can be made by each, an intelligent comparison can be made.

In the construction of concrete bridges, culverts, or walls, this may be by all means the best way to do the work, because it very often occurs that the engineer desires certain kinds of rods, placed as he plans them, or certain brands of cement or other material, which he wishes

Mr. Parker.

particularly to have embodied in the work; and, during the work, or at its conclusion, this method would prevent the question arising as to the use of other brands of material which might justly be claimed to be of equal quality, thus creating controversy between the parties to the contract. At any rate, in cases where it is particularly desired to use materials of a fixed type or character, and where there may be doubt in the minds of experts generally as to which is the best, thereby producing uncertainty in the interpretation of the specifications, there can be no question, if the materials are supplied by the party of the first part, and the labor alone by the contractor. This is so true that the Highway Commission of Massachusetts has for many years followed this practice in the construction of its concrete structures, and the reasons for this appear to be good. At the beginning of each year, the Commission receives bids for cement, to meet certain specifications. This cement is tested at different times, in order to see that it comes up to the requirements. It is furnished to the contractors as it is needed, and their responsibility for its quality is not a part of the contract. In this way, uniformity of material and satisfaction are ensured. The contractor is also supplied with rods, of dimensions determined by the engineer, and these are placed as he directs. The contractor in this case furnishes only the labor and the gravel or broken stone.

Under the third method, where both labor and materials are supplied by the party of the first part, the instances where contracts are let can be but few, if, indeed, it may be said that a contract can be entered into where neither material nor labor are furnished. This method, which, from the nature of things, can involve no agreement between parties, would be preferable in the performance of work of a character which it is impossible or impracticable to define in specifications with sufficient clearness or precision to enable one to make an accurate estimate of the cost. In the performance of such work it is also preferable because special skill and knowledge are necessary, and these have not been acquired by contractors, or, at any rate, by a sufficient number of contractors to secure fair, competitive bids.

From the speaker's experience in Massachusetts and elsewhere, he believes that work under contract should be done under either method *"a"* or method *"b."* If method *"c"* is contemplated at all, it should be when the conditions are such that reasonable bids cannot be obtained, on account of the inexperience of contractors, or for other reasons. Therefore, method *"c"* only contemplates a comparison between contract and day labor.

In order to get economical work by day labor, it is necessary that the party of the first part shall have a constant force of laborers and foremen trained in the particular kind of work he has in mind, and constantly employed on work of the same type or class.

Mr. Parker.

It may be, and, in fact, is found, that in the maintenance of certain roads, where the conditions under which a contract could be made are uncertain and involve unknown costs which will not permit the contractor to make a reasonable bid, method *"c"* is preferable.

In maintaining roads in Massachusetts, especially where oil and tar are used, it has been found that the work can be done more economically by the third method than by either of the others; and it is better done, because the maintenance gangs are well organized, and are under trained foremen, who have learned all they know under the direction of the engineers of the Commission, and, from their practice under very careful supervision, have become expert. Consequently, the work is better done and at a less cost than it could be by letting it to any contractor.

It seems to the speaker, therefore, that, where the conditions warrant, any one of the three propositions might be adopted by the party of the first part, but it must be for that party to determine, after most careful consideration, which of the three is best suited to the work in view.

In general, the speaker maintains that, wherever possible, method *"a"* (the contractor furnishing all labor and material) is preferable for all construction, except where reinforced concrete constitutes a large proportion of the work.

Method *"b,"* in which the contractor supplies the labor and the party of the first part the material, or a certain proportion of it, is preferable where the largest portion of the work is of reinforced concrete, or is of an experimental nature, or where a bituminous material of a specified kind is used.

Method *"c,"* in which the party of the first part furnishes both the labor and material, is preferable in cases where the amount and kind of work are difficult to determine, and where it is of such a nature that the contractors cannot make intelligent estimates of the cost.

Mr. Browne.

HENRY B. BROWNE, ASSOC. M. AM. SOC. C. E.—This discussion pertains to the relative efficiency of three methods of carrying on bituminous road construction. For several reasons, the arrangement between the contracting parties is not as satisfactory under the first method as under either of the others. The majority of road contractors are inexperienced in bituminous construction, and, therefore, the bids received are generally out of proportion to the actual cost of the work, and in most cases allow the contractor an unreasonable profit. In doing work by this method it is necessary to have the specifications covering the bituminous material ready at the time the contracts are advertised, or a list of the different brands of material on the market which will be allowed in the work must be given to the contractor. If the contractor selects the material from the approved list, the cost of all rejected material will have to be borne by the party of the first

Mr. Drowne.

part, and, furthermore, as bituminous materials manufactured to-day and given the same trade name vary so much in quality, the party of the first part would be taking a great risk of not obtaining good results in so far as they may be affected by the material. On the other hand, if the bituminous material is described in the specifications, it may be found, during the progress of the work, that, although it has fulfilled the conditions imposed, it is not at all satisfactory, and a change may be desirable. These changes may affect the characteristics of the material, so that the cost of manufacturing such a product may be considerably less than for that first specified. Although the contractor is generally willing to agree to any changes in specifications which increase the cost of the work, provided due allowance is made for the same by the party of the first part, it is unusual for him to agree to any reduction in his contract price, although it can be clearly proven that any proposed change in specifications would reduce the cost of the bituminous material which he is obliged to purchase. The same remarks are true in regard to the specifications as to the method of doing the work. As far as the rate of progress of the work is concerned, the speaker feels that, although more actual yardage may be obtained per day when the work is let by contract, the quality is apt to be sacrificed, as the contractor is much more interested in the quantity than in the quality. Extreme care in doing the work is one of the essential elements of successful bituminous roads.

The only advantage that the speaker can see in performing the work entirely by contract is the saving in office labor that would ensue if the party of the first part furnished the bituminous material and had the work done on labor account.

When all road contractors have become familiar with bituminous road construction, and the engineers are so well informed that they can specify a certain bituminous material and method of construction and know that the resulting road should be good for a definite number of years, then the work could be let out by contract, and fair prices would probably be obtained. If the party of the first part employed intelligent and competent inspectors, careful work on the part of the contractor would be assured. The use of a guaranty clause in the contract, however, is rather a doubtful expedient for securing good work. A poor piece of work can sometimes be kept in repair for the number of years stipulated in the guaranty at a small cost, but at the end of this period the road may need such extensive repairs that a renewal would be necessary. Moreover, the cost of the maintenance for the years covered by the guaranty has been paid by the party of the first part, because the contractor has made allowances for this in his bid.

The speaker has had considerable experience with the second method, in which the bituminous material is supplied by the party of the first part and the labor by the contractor. The contracts were

advertised as ordinary macadam, and bids were received for stone in place and grading. If it was decided after signing the contract that the work was to be of bituminous construction, the contractor was informed, and a separate agreement was drawn up and signed by both parties. This agreement covered the prices for labor furnished, all allowances for equipment, etc., necessary for the bituminous construction, and any deductions that were to be made from the contract price for work to be done on labor account and already included in the contract. Although in no case did any contractor refuse to sign the agreement, some were reluctant to agree to any deductions. The party of the first part paid the contractor the amount of the labor bills for the bituminous work, minus whatever deductions were agreed on, plus a 15% allowance on the remainder, and plus the allowances for equipment. Mr. Drowne.

Different methods of construction were tried, and on one road both the method of construction and the material were changed several times, with no objection on the part of the contractor. It seems to the speaker that reserving the right to change anything in connection with the bituminous work without having any trouble with the contractor is a great advantage. The party of the first part, in purchasing the bituminous material direct, saves any profit which might have been made by the contractor if he had purchased it, and by paying the actual cost of the work plus a 15% allowance, it can be expected that the cost will be as low as possible.

The rate of progress, by doing the work in this way, is probably not as great as if done by contract. The labor, however, is not the large item in the total cost of bituminous construction, and so a fair rate of progress with the work done in a thorough manner should be satisfactory. A comparison of this method with that of doing the bituminous work by contract came under the speaker's observation two years ago. In the first instance, the contractor had built a bituminous pavement by contract for 22 cents per sq. yd. over and above the cost of ordinary macadam. In the second instance the conditions were exactly the same as in the first, in so far as the parties to the contract, location of road, method of construction, and materials were concerned, but the bituminous material was furnished by the party of the first part, and the contractor did the bituminous work on labor account. The cost of the work in the second instance was 12 cents per sq. yd. over and above the cost of ordinary macadam. The price for the ordinary macadam was determined by contract. An allowance was paid to the contractor for the use of the extra equipment for the bituminous work (consisting of two kettles, fuel, mixing boards, dippers, and long-handled shovels), and also a 15% profit on the labor bill. For this work there were no overhead charges except on the extra equipment involved. The allowance made for this by the party of the first part more than paid the actual depreciation, and made the 15% on the

Mr. Drowne.

labor practically clear profit. The price of 12 cents includes everything mentioned above, and is, therefore, the true cost which can fairly be compared with the cost of 22 cents.

Some contractors object to this method, mainly because the work may be slower than if ordinary macadam were built. For example, on a pleasant day immediately following a rain it is possible to do ordinary macadam road work, but it is not possible to do any bituminous construction. The bituminous gang has to be paid at such times in order to keep it, and if the men work, it is oftentimes at a disadvantage. With the better class of contractors, this method has worked well, and the time for completion of construction has not been much longer than if ordinary macadam had been built.

It is the speaker's belief that this method is the best of the three, provided that, if a bituminous road is to be built, it is advertised as such, and that some clause is added to the extra work clause in the general specifications to include the bituminous construction as extra work, subject to agreement between the two parties. If such a course is followed, the method is fair to the contractor, and should result in good work at a low figure.

The advisability of using the third method, the party of the first part furnishing all material and labor, depends to a great extent on conditions. Excellent results ought to be obtained by having the work done by a gang of men trained specially for the purpose. As the party of the first part has also an unhampered choice of materials, there is no reason why those best suited to the road could not be obtained. There is no doubt that maintenance work and the application of a bituminous surface to roads already constructed could be accomplished economically by this method, as these particular branches of the work do not require either large gangs or much equipment, and the rapidity with which the work can be done allows a few gangs of trained men to cover a large territory. Any large amount of new construction in one season could not be carried out by the smaller municipalities and towns, on account of the large equipment required to do the work. In places where an efficient public works department is maintained and in other places where sufficient work is contemplated to warrant an outlay for the necessary equipment, this method might be used to advantage.

If the bituminous work was to be done by the mixing method, the speaker believes that it could be done economically by establishing bituminous mixing plants at points central to the different jobs, and by shipping the mixed stone ready to be put into place. There are several machines now on the market which can do a great deal of work provided they are properly set up and operated and the aggregate is furnished in quantities commensurate with their capacity.

The success of the third method, if conditions warrant its use, depends to a large extent on the efficiency of the corps of laborers and foremen employed by the party of the first part. Politics creep in on most public work, so that oftentimes many incompetent men

are employed, and, although civil service remedies this evil, it is almost impossible to eliminate it entirely. Many instances might be cited where the labor cost in doing the work by this method is as cheap or cheaper than if the work was done by contract, and still, as a general rule, the opposite is true. Mr. Browne.

JOSEPH W. HUNTER, M. AM. SOC. C. E.—The Highway Department of Pennsylvania has had very little opportunity to determine many facts concerning this subject. The act under which the department is working practically specifies how the contract shall be made. It has been found, however, that in the long run it would pay the department to furnish the material and have the work done by contract, paying a fixed rate per hour for the men and for the use of such machinery as might be required. For instance, in the case of making repairs to a road many miles in length, bids were received for doing the work in the usual way, amounting to about $4 000. An actual account of the time for doing the work showed that if it had been done by the contractor furnishing the labor and the department furnishing the material, and allowing 15% profit to the contractor, the saving would have been a little more than $1 300. It seems to the speaker that, if the department had the authority to do most of the work by this method, it would be satisfactory, not only to the department, but to most of the contractors. The quality of the work would probably be as good as is now obtained under the contract system, if not better. If a contractor, in bidding for work, makes a mistake or error in his bid, there is generally some trouble in coming to a final settlement; but, in doing the work by this method, the contractor would be in a position to suffer no loss at all. The matter of furnishing both labor and material has never been attempted by the department in any way. Mr. Hunter.

W. W. CROSBY, M. AM. SOC. C. E.—The speaker wishes to state at the outset that he believes in making contracts wherever it is practicable. Where one man is in charge of a large amount of work, it is easier for him to carry it along under definite specifications. Mr. Crosby.

Most engineers want to be fair. In order to be fair, they must know the facts, and they should not use incorrect figures or distort facts, even unintentionally, in drawing comparisons. A great many instances of cost by force account are reported, however, where it is evident that proper consideration has not been given to such things as "overhead charges," which a contractor must figure into his prices.

In most American laws regarding the execution of work and the expenditure of money, with which the speaker is familiar, there is generally an opportunity to do some work by force account. In Maryland the department has opened many bids on miles of roads far away from the centers, and has rejected them because they were high. In one case the bids averaged about $10 000 per mile, and the work was done later by force account at a cost of less than $7 500 per mile.

Mr. Crosby. Unquestionably there are instances where any one of these three methods of doing the work can be used, and the speaker thinks the method to be followed depends largely on local conditions. In the case of experiments, it is quite desirable to have a free hand in selecting the material with which experiments are to be made. It is undoubtedly a relief to both parties in some instances if the party of the first part furnishes both labor and material; but, even then, a contract—or an agreement in the nature of a contract—covering the labor and the rates for it, may be made.

Mr. Sargent. Paul D. Sargent, Esq.*—All these methods have been used with satisfactory results in Maine. Practically all the bituminous work has been done under the second method; ordinary macadam has usually been constructed under the first method, but only two or three roads have been built by the third method.

Considerable difficulty was experienced in Maine in 1910 in getting contractors to bid on the State roads, because there was so much work of that kind going on in New Hampshire and Massachusetts. This made it necessary to re-advertise several jobs two or three times, in order to obtain satisfactory bids. For one road, 2 miles long, to be built of gravel, bids were advertised in July, and the lowest figure was $13 900. Perhaps it would be fair to state that there was only one pit from which gravel could be taken; and that, to the best of the speaker's knowledge, the contractor who put in the low bid was very busy, having three or four large jobs under construction. The speaker thought this bid was too high, and arranged to do the work by the third method. A State road inspector was put in charge, and he had two foremen, one at the pit, at $2.50 per day, and one on the road, at $3.00 per day. The road constructed was at least 50% better than the original specifications required. The gravel surface was 18 ft. wide, instead of 15, as originally called for, and the depth of gravel after rolling was from 10 to 16 in., instead of 8 in., as originally planned. Moreover, 700 ft. of the road was surfaced 21 ft. wide with trap-rock macadam. This stone came from Massachusetts, and cost $1.20 per ton, delivered on cars. It had to be hauled about ¾ mile. The total cost was $5 850. This includes a portion of the inspector's salary, the cost of all hand-tools used on the work, and also a blacksmith's bill of about $30 for sharpening picks. The road was rolled with a horse roller. No charge is included for the rent of the roller or watering cart.

Mr. Ellis. Fred. E. Ellis,† Esq.—There are two classes of material which the contractor would be very glad to have the party of the first part furnish, namely, gravel and local stone. It is surprising how rapidly

* State Highway Commissioner of Maine.

† Manager, Essex Trap Rock and Construction Co.

the value of these materials increases when a number of contractors are figuring on the same job, and they are all bargaining with the same man. It seems to the speaker that, if the engineer could secure options on gravel and local stone, it would decrease the cost of the road materially. If this could be done by the party of the first part the contractors would all stand on the same footing in regard to these materials. The purchase of cement and bituminous materials does not affect the contractor as much, because both can be purchased from a number of sources and there is some competition. Mr. Ellis.

When gravel and local stone are not furnished by the party of the first part, it is possible for a contractor to corner the supply (and in some cases it has been done) and thus exclude other contractors from making a proposal for the work.

A contractor very often is asked to pay more for gravel and stone after a contract has been awarded to him than the price agreed on verbally before making the proposal, and in most cases is obliged to pay the increased price, as the supply is generally limited to the material owned by the party asking the increased price.

These uncertain conditions would be avoided by the party of the first part securing options on materials of this class.

Samuel Whinery, M. Am. Soc. C. E.—In some kinds of street pavement and road construction it would undoubtedly be advantageous for the authorities in charge to supply at least some of the materials required. This is especially true of bituminous materials, which are offered on the market in such great variety and differ so much in quality that careful examination is necessary before they are used. This examination requires more time than is usually available where the materials are supplied by the contractor and, as is often the case, are delivered on the work but a short time before they must be used. Mr. Whinery.

The good quality of the bituminous cement used, and its suitableness for the particular conditions to be met, is one of the most important elements in modern road construction.

It is impossible to prepare specifications which will be sufficiently definite and specific, and, at the same time, will admit wide competition, for the reason that bitumens which may be equally suitable for the purpose, if properly used, may differ quite widely in their chemical and physical properties. For instance, in the construction of sheet-asphalt pavements, it is usually attempted to frame a set of general specifications in such a manner that any one of several different kinds of asphalt may be used. In order to do this, it is necessary to provide such wide latitude in the requirements for the bitumen that the specifications when applied to any one kind of asphalt are loose and indefinite. In other words, the door is purposely made so wide in order to secure competition that almost anything may claim admittance. If, in such a set of general specifications, the requirements are made sufficiently

Mr. Whinery. specific and detailed to apply to one kind of asphalt, another kind equally good if properly used may be excluded. This is true, not only of the asphalt itself, but of the work in which it is to be used, as the proportions of the mixture, the consistency of the asphaltic cement, and the permissible temperatures, all vary more or less with the kind of asphalt used.

There is only one satisfactory remedy for this state of affairs, and that is to prepare the road construction specifications on the basis that some one particular kind or quality of asphalt will be used. This condition may be secured in either one of two different ways, first, one particular kind of asphalt may be designated for use, and specifications framed accordingly. This method is objectionable because it may restrict competition within very narrow limits. Second, the bitumen may be purchased under general specifications for this material alone, the most favorable material for the purpose, considering both quality and price, selected, purchased, and stored, to be supplied to construction contractors at a stated price. Under this method, proposals covering a wide variety of bituminous materials could be called for, accompanied by samples; these samples could be thoroughly examined in the laboratory, and, in due time, the most advantageous material selected.

Later, in preparing the construction specifications, these could be framed with particular reference to the material selected to be used. The wide exercise of judgment and discretion generally allowed the contractor in using his own material under the present system could thus be eliminated, and strict conformity to definite specifications enforced.

That such a system would result in better work cannot be doubted, and while the re-handling and re-shipment of material would add to its original cost, this increased cost would almost certainly be overcome by the lower price at which the material could be purchased in large quantities at seasons of the year when the market was not overcrowded. This plan would remove the uncertainty about the quality of the bituminous material which goes into much of the present road work because of the lack of time and facilities for careful inspection.

Mr. Pullar. H. B. Pullar,* Esq.—The best roads are laid by either the second or third methods, because the material is furnished by the party of the first part. Contractors naturally wish to make as large profits as possible, and so they use the cheapest material they can obtain. Manufacturers are obliged to make such materials in order to compete with others for the business. On the other hand, where the party of the first part buys the material and does the work by labor account, better results can be obtained, and, although the work may go on more slowly,

* Chemist of the American Asphaltum and Rubber Co., of Chicago, Ill.

the details are looked after more carefully, and better materials are likely to be used because the contractor is not responsible for them. Mr. Pullar.

There is one point on which most producers will agree, namely, that it is unfortunate that different ideas are held by engineers in regard to the requirements of bituminous material to be used for the same class of work. One engineer will want one kind of material, and another will want something entirely different. Of course, it is impossible for any producer to manufacture one material which will meet the requirements and personal ideas of all engineers, especially when the conditions are changing so much that the materials used in roads laid two years ago are very different from those used to-day; and probably they will be changed again two years hence, if the roads laid last year do not prove to be satisfactory. The speaker thinks that engineers and manufacturers of bituminous materials should co-operate to secure the standardization of the materials and the methods of testing them.

Nelson P. Lewis, M. Am. Soc. C. E.—The speaker believes that where there is a proper organization and laboratory equipment it would be wise for the municipality, the county, or the State, to purchase the materials to be used, and then make a separate contract for the performance of the work in accordance with specifications drawn to suit those materials. Satisfactory results can only be attained by the careful selection of materials, an intelligent specification for their use, and honest, intelligent workmanship with these materials in accordance with the specifications. Substances which are apparently similar, but with different qualities, require different manipulation, and this is conspicuously the case with those used in modern highway work. The actual labor can readily be done by contract under competent supervision, but if the contract also includes the furnishing of all the material, it is exceedingly difficult to frame in advance a specification which will insure the best results with such materials as may be offered by the lowest bidder. The speaker's experience is not such as would encourage an attempt to perform the actual construction work by day labor. There is no reason why asphalt and other bituminous materials, including oil, should not be purchased in large quantities, a contract for their use—and describing the precise manner in which they are to be manipulated—being prepared after the peculiar qualities of the material have been ascertained. This could be done with care and deliberation, and it is fair to assume that the ultimate results would be more satisfactory. Mr. Lewis.

The necessity for organization in the maintenance of highways has been amply demonstrated, and an object lesson as to its benefits has been furnished recently in the State of New York. As is well known, this State proposes to spend very large sums in highway construction, and a few years ago there was grave danger that this money might be

Mr. Lewis. expended in the construction of additional roads before a thorough study had been made as to their location and before reaching a decision regarding a proper system of maintenance. The policy followed by the present Highway Commission—of deferring the letting of contracts for a large additional mileage until thorough investigation has been made and a careful plan for maintenance has been evolved—must commend itself to every one. Whatever changes may be made in the organization controlling the highway work of this State, it is to be hoped that the progress which has been made during the last few years will be maintained.

Mr. Parker. HAROLD PARKER, M. AM. SOC. C. E. (by letter).—This subject is one which undoubtedly has excited a great deal of interest, and, being a comparatively new art, as far as American engineers are concerned, it has received the attention of many of the leading men engaged in the construction of highways.

The relation between the contractor and the engineer is always of the utmost importance, and, as far as highway construction is concerned, is more or less under trial, much more so than in any other kind of construction work. This is true because the art of road-building is comparatively new, as stated, in America, and also because the introduction of bituminous material in the top course of a road or on its surface is only a partly explored field.

For this reason, and because the contractors are unable to determine, by experience, the cost of doing this work, the relation between the contractor and the designer is somewhat different from that in ordinary work. It might well be, therefore, in the interests of good results and economic expenditure, that the materials be supplied by the party of the first part, and the labor by the contractor.

From the writer's experience, and from what the discussors have found to be true, there appears to be no doubt that either the second or third methods must be used. This is also indicated, as in all work of more or less experimental character, by the fact that, where doubt exists as to the cost, the contractor must put his price at the highest figure. This doubt may not exist after the contractor, or party of the second part, has had enough experience, both in the character of the material to be used and in the method of applying it, to determine a proper competitive figure.

Up to the present time, the writer has found that better work can be done for less money by furnishing both material and labor, that is, by obtaining the necessary material by specifications and trials, and by having gangs of trained men, with foremen experienced in the particular kind of work to be done. This statement is limited to some extent, of course, because any work of very great magnitude could hardly be managed in this way. In the present state of the art, however, this seems to be the most economical way of carrying on bituminous work.

(3) SYSTEMS OF MAINTENANCE.

By Messrs. Hubert K. Bishop, George C. Diehl, R. A. Meeker, James H. MacDonald, and Logan Waller Page.

Hubert K. Bishop, M. Am. Soc. C. E.—Instead of trying to give a general discussion on systems of maintenance, the speaker will confine his remarks to a brief description of the organization and methods used during the past two years in New York State. Mr. Bishop.

When New York first began the improvement of roads, the idea prevailed that when once constructed they would last for all time. Experience, and changes in traffic conditions, have altered this idea, and the general public recognizes that an improved road is subject to depreciation, and must be maintained and repaired, the same as any other physical property. The importance of proper attention to detail, and the necessity for prompt repairs and renewals cannot be overestimated. There is no economy in neglecting macadam roads, whether they are water-bound or of bituminous construction. On the other hand, inattention to proper repair and maintenance is the wildest extravagance. It would be better to curtail the number of roads improved than to neglect the faithful care and maintenance of those already constructed.

The 2 000 miles of improved roads in New York at the beginning of 1909 were in a deplorable condition, and this was due, partly to faulty construction, but mainly to neglect and lack of proper maintenance. Previous to 1909 there had been no systematic plan of maintenance; appropriations by the Legislature had been inadequate, and what little money had been provided was expended in the scattered and not always effectual work of re-surfacing. After a careful study of conditions, and the little information available on this subject, it was decided to establish a separate organization, distinct from the construction department; and that, in order to secure satisfactory maintenance and repair, it was necessary for each man to give his undivided time and best efforts to the work. It was also thought to be extremely important that each man should have a particular territory, and be responsible to some one over him.

In the beginning of 1909 the improved roads were widely scattered, being distributed in more than 50 counties and more than 500 towns, covering a territory approximately 500 miles long and 350 miles wide. The State was divided into six divisions, and each of these was subdivided into from five to seven sections, according to the mileage of road in each division, so that approximately 60 miles of improved highway were included in each section. A Division Superintendent was selected from the construction force of each division. In making

Mr. Bishop.

this selection, every endeavor was made to obtain men of broad ideas and of large experience in the construction and maintenance and repair of improved roads, and also men of tact and good judgment. The improved roads of each section were divided into patrols of approximately 5 miles, depending entirely on the kind of pavement to be patrolled: In the case of brick roads, the length of patrol would be 10 or 15 miles, in the case of asphalt roads, constructed by either the penetration or mixing method, about 8 miles, and in the case of straight water-bound macadam roads, about 5 miles. It is not always possible to do this, because the roads are scattered, and because sometimes there may be a road 2 or 3 miles long in a locality where there is nothing to add to it. Highway inspectors, from the force of employees of the State, were selected for their general qualifications and experience in road work, and headquarters were established in each division for the Division Superintendent.

The men selected as patrolmen were from the localities in which the roads were situated, careful inquiry being made as to their character, habits, faithfulness, and willingness to work. Each applicant for these postions was interviewed by the highway inspector before being employed. Each patrolman is required to furnish a horse, wagon and the necessary small tools to perform such labor as may be required by the highway inspector in charge, and each is required to spend all his time, between the hours of 8.00 A. M. and 5.00 P. M., patrolling and working on the patrol assigned to him. Each patrolman is furnished with a metal staff, consisting of a round disc, approximately 6 in. in diameter, attached to a steel rod about 3½ ft. long. On this disc is painted the number of the patrol, and he is required to place the disc in the shoulder of the road at a distance not exceeding 200 ft. from the point at which he is working. If he is obliged to leave the road, to get gravel or stone, or for any other purpose, he places the disc at the side of the road so that the highway inspector may know where to look for him on his return. A patrolman is often furnished with additional labor and teams, and acts as a foreman, making minor repairs under the direction of the highway inspector in charge of the section. Each patrolman is responsible to the highway inspector for the condition of the road in his patrol. The highway inspector travels continually over the patrols in his section, at irregular intervals, and instructs the patrolman in the manner of making the necessary repairs to prevent the deterioration of the road. He also checks up the patrolman to see that he is performing his work satisfactorily. The patrolman reports weekly to the highway inspector, on a postal card form, writing opposite each day of the week (printed on the card) the station at which he worked, and the nature of the repairs. The highway inspector, in turn, makes a weekly report to the Superintendent of Repairs, showing his movements for each day in the week, roads

inspected, instruction given to patrolmen, and repairs being made. The Superintendent of Repairs visits each highway inspector in his division as often as possible, takes up with him the question of the repairs necessary to each road in his section, advises him how to make such repairs, and gives him the necessary authorization to expend money on the same. The Superintendent of Repairs reports daily on a postal card form to the First Deputy, at Albany. His report gives the number of roads inspected each day in the week, their condition, and his advice as to the necessary repairs which he intends to carry out. Mr. Bishop.

In order to have a more thorough check, and be certain that each man is attending to his duties, an inspector at large, who reports directly to the Highway Commission, and who is unknown to the Bureau of Maintenance and Repairs, travels through the various counties of the State, interviewing patrolmen and checking up the highway inspectors. He submits detailed reports of his findings directly to the Commission, who, in turn, submits them to the First Deputy for investigation and report. When one of these reports shows that a patrolman has been absent from his beat, or that the highway inspector has been neglectful of his duties, the man in question is promptly discharged.

The work of the patrolmen is to eliminate ruts, trim shoulders, paint guard-rails, open waterways, cut grass, apply screenings or gravel to prevent ravel, and spread oil and stone on roads which have been oiled; in fact, it consists of all such minor repairs as the man is able to do on the number of miles assigned to him. Other repairs, of a minor nature, but too large to be handled by the patrol system, are done by contract. The highway inspector in charge of a section superintends this work, and it usually consists of oiling, and the reconstruction of guard-rail and masonry structures. The work of resurfacing, or renewal of the top surface, is done by contract.

The patrolmen are employed for nine months of the year, and receive from $3.00 to $3.50 per day. The highway inspectors are employed for the full year, and receive $4.50 per day and necessary expenses. During the three months in which the patrolmen are not employed, the highway inspectors are engaged in making out estimates for the repairs for the following year, which are revised by the Superintendent of Repairs, and, in turn, by the First Deputy; they also take inventories of material, lay out the next season's work, prepare necessary contracts, prepare cost data and maps which show the location of all available gravel pits and stone quarries, and such other work as may be required of them. The Superintendents of Repairs are likewise engaged.

The annual cost of the patrol system, as separated from the other work of repairing, is practically $150 per mile, including the cost of supervision. The cost of the patrol itself, independent of the cost of

Mr. Bishop. supervision, was $106 per mile in 1910. In 1911 it will be more, because more patrolmen have been engaged, and the length of the patrols has been shortened. From 25 to 50 cu. yd. of stone, screenings, and gravel per mile are required for maintenance material per year. This would cost, including the oil for repair work, from $50 to $150. The cost of the patrol is not the whole cost of maintaining the roads of the State, but is only for the work done under that system. Other work is done, such as making improvements, re-capping, building culverts, and other minor repairs. In New York State the annual cost of maintaining a water-bound macadam road, in the broad sense, is approximately from $350 to $450 per mile, while the cost of an asphalt macadam, well constructed, is less than $200 per mile.

The best investment that New York State has made is the patrol system, or the plan of constant attention to small detail. Roads which were in bad condition are hardly recognizable after a few weeks' work of the patrol. In the speaker's opinion, with the amount of money invested, it is possible to do more by the patrol system than in any other way, and at present he is thoroughly convinced that, with that system, plenty of maintenance material, and occasional surface oiling, the life of the roads of the State can be materially lengthened, and the annual cost of depreciation can be cut in half. It would seem that the important features of a successful maintenance system, assuming that sufficient funds are available, are that each man employed in this work should be responsible for a definite territory, and that all his time should be given to familiarizing himself with his territory and his work. Each man should be wholly devoted to the subject of maintenance and repair, as distinct and separate from the work of construction. It is desirable, of course, that these men should have had experience in construction before taking up maintenance and repair. It is also important that the whole system should be founded on merit. If such a system is adopted, each employee becomes more and more familiar with the territory under his charge, and consequently more valuable to the State.

Mr. Diehl. George C. Diehl, M. Am. Soc. C. E.—To a great extent, experience must be the guide in road construction and maintenance. Mr. Bishop has described the patrol system of maintenance on State and County roads in New York State, and it might be well to learn the views of the users of the highways regarding this method of maintenance. A somewhat intimate relation with several farmers' organizations, and also with the American Automobile Association, has given the speaker an opportunity to obtain expressions from thousands of users of these highways, and it appears to be the unanimous opinion, both of the users of horse-drawn vehicles and of the drivers of motor vehicles, that the State has received, dollar for dollar, value from the appropriations for the patrol system and the expenses incident thereto: and they

believe that that system provides an efficient, economical, and rapid method of preserving, maintaining, and repairing the principal highways. Mr. Diehl.

These highways are divided into three classes: State, County, and Town. The first and second comprise about 10% of the total mileage, and are the only roads maintained under the patrol system. The town or lateral highways comprise about 90% of the total mileage, and for many years to come, not only in New York, but in all the States of the Union, must form the preponderating mileage of public highways.

Therefore it might be appropriate to say a word regarding the method of maintaining lateral roads in the State of New York. The expense of maintaining such roads is borne jointly by the township in which the road is located and by the State, approximately in the proportion of two-thirds and one-third. The State Highway Commission has control of these expenditures, but the work is performed by the chief township highway officer. The State Highway Commission insists that not more than one-fourth of the moneys available shall be expended in the primary or temporary work of scraping and ditching. This permits the bulk of the money to be expended in permanent work, such as the construction of concrete culverts and bridges, widening, reducing excessive grades, and placing permanent road metal. The result is that the lateral roads grow better year by year. Up to date, 2 300 miles have been macadamized, 8 000 miles graveled, and 53 000 miles shaped and crowned, as compared with 2 400 miles of State and County roads constructed and maintained by the State Highway Commission.

This is in striking contrast to the work done before the present system was introduced, and also to that done in many other States where all the township highway money is expended in temporary work, with the result that from year to year the roads show practically no improvement. Some system such as this must be inaugurated before it can reasonably be expected that the 2 000 000 miles of public highways throughout the United States will be cared for properly.

Highway engineers at the present time, by reason of the newness of the movement, are called on, not only to design and carry out road improvement and maintenance, but to advise as to the best methods of organizing highway departments, and in many instances to organize movements which will arouse public sentiment in favor of comprehensive highway legislation and liberal appropriations to make such laws effective.

It has been said that politics and road construction do not go hand in hand, but that they must be separated as far as possible. Under present circumstances, highway engineers should not be backward in expressing their opinions. In order to secure the best results, con-

Mr. Diehl. tinuous policies are desirable, and, consequently, the highway work should be done under the supervision of long-term appointive officials, rather than short-term elective officials. There is now a movement on foot in the State of New York which seeks to transfer the highway work from such long-term officials to short-term elective officials. It is believed that this movement, if successful, will have a tendency to inject politics into the road movement and to discourage continuity of policy and a reasonably long tenure of capable engineers in office. This statement, it is believed, is equally true regarding town, county, and State highway officials. To secure the best results, in both construction and maintenance, there should be centralization of authority. There must also be large appropriations, because, like all other public work, the improvement of highways properly done is expensive.

It has long since been demonstrated that the townships in which the roads are situated are not of themselves capable of sustaining the entire expense of road construction, and there has been a movement all over the country to enact county and State aid laws. The principle on which these laws are based is that the cities or urban localities deriving considerable benefit from the highways should share with the rural communities a portion of the burden. In some localities county aid laws make a proper division of this expense. In some States—notably the Eastern States, containing the great centers of population—State aid laws work very satisfactorily, but there are many States which do not contain great cities, where even the State will form too small a unit for the proper division of the expense of road construction and maintenance, and, before suitable systems of roads, comparable with foreign systems, can be built within the United States, it is believed that Federal legislation and aid for highway construction must be obtained.

Mr. Meeker. R. A. Meeker, Esq.*—In New Jersey the organization for maintenance is composed of the commissioner of public roads, the supervisor of roads, two assistant supervisors, and in each county a salaried county supervisor, who is appointed for a term of three years, and whose duty is to take care of the county roads. The county supervisor is required to keep his roads in proper condition, and he may be dismissed for failure to do so.

There is a fund amounting to about $350 000 a year from the licenses obtained from automobiles, which is all expended for the maintenance of county roads under the supervision of the State road department throughout the several counties.

The township roads are maintained by the township authorities. The road tax is not worked out in New Jersey, but is paid in money, and this is expended generally by the committees of the several town-

* State Road Supervisor of New Jersey.

ships, with indifferent success, on the township roads. Conditions are improving, mainly because each township is endeavoring to maintain its roads in better condition than the adjoining one. Mr. Meeker.

JAMES H. MACDONALD, ESQ.*—In Connecticut there is a county system of supervision of maintenance. In each of the eight counties, there is an inspector whose duties consist in seeing that the State roads are kept in repair. In addition to these inspectors, there is a deputy engineer for each county, who, in addition to looking after the work of making surveys and exercising an oversight of the work of the local inspectors, assists in the matter of repairs. Mr. MacDonald.

This system was inaugurated only two years ago, but prior to that time, for a period of thirteen years, the towns were responsible for the repair of State roads. The former system of town repairs met with only indifferent success. While the men are new at their work under the patrol system, the results have been very satisfactory, and it is hoped that as those in charge become more thoroughly acquainted with the system, the State will secure a splendidly organized corps of men to take care of the question of repair and maintenance.

LOGAN WALLER PAGE, M. AM. SOC. C. E.—The speaker is much interested in the small cost of maintenance in the State of New York under the patrol system. In England and Wales, where the work is not under the patrol system, the cost of maintenance for the main trunk line roads last year was approximately $440 per mile. In France, under a centralized system, the annual cost was about $220 a mile. This matter of maintenance is the most important part of the whole road problem. Mr. Page.

State road work is quite new in the United States, and, in agitating the subject, a great deal has been said about the building of permanent roads, the average citizen—and legislator, in particular—having the idea that when a macadam road is built it is permanent. Few of the States do any real maintenance work, and in some cases the repair work is done with borrowed money. It is essential under certain of the State laws to scatter the work, a mile here and a mile there, in the different counties of the State, so that it is almost impossible to introduce the patrol system.

There are 2 155 000 miles of county roads in the United States. If 10% of these roads were to be constructed, it would probably cost about $2 155 000 000; it will be a very long time before any such sum of money will be available. For many years this enormous mileage of roads must be maintained under the present system of county supervisors, and these men, to a large extent, are political officials. In one county, of which the speaker knows, there are more than 180 supervisors and no laborers. There are more than 100 000 paid county

* State Highway Commissioner of Connecticut.

Mr. Page. road officials in the country, and they are expending now about $90 000 000 a year on the public highways. In the speaker's opinion, only a very small percentage of this money goes to make the roads better than they were the year before. There is no system about it. Earth and stone are dumped on the roads in the fall and in the spring, and the traffic avoids the road as long as it can. The whole situation is most discouraging. If more attention could be given to the byways of the country, it would result in a great amount of good.

(4) THE USE OF WATER, CALCIUM CHLORIDE, LIGHT OILS, ETC., AS DUST PALLIATIVES.

By Messrs. S. Whinery, J. L. Wickes, H. C. Poore, Charles W. Ross, Hubert K. Bishop, L. R. Grabill, and R. A. Meeker.

Mr. Whinery.

S. Whinery, M. Am. Soc. C. E.—In the discussion of this topic the element of road preservation is excluded, except in so far as the use of the materials described may incidentally affect that problem. The field for discussion, therefore, is a narrow one, confined principally to the relative merits and economy of the various materials for allaying dust.

The best known and longest used method of making and keeping streets and roads dustless is by sprinkling with water. At the same time it may be said to be the least developed process, as far as scientific study and application are concerned. Accurate and extended observations on the effect of sprinkling roads and streets, the cost of the process, the quantity of water required, the best and most efficient method of applying the water, and other pertinent matters have either not been made or the results have not been published. If any one doubts this, let him attempt to obtain, from the scanty literature relating to the subject, reliable and consistent data on any of these points.

Municipal engineers seem to have always looked on street sprinkling as an every-day, matter-of-course operation, not worth serious attention and study, and not susceptible of much improvement in efficiency and economy. It is safe to say that no other municipal function or process receives so little intelligent attention and supervision.

When the sprinkling season arrives, the usual programme is to equip a more or less unintelligent and uninterested driver with a sprinkling wagon and a hydrant wrench and turn him loose to work his sweet will on the street to which he is assigned. His autocratic reign is only interfered with, and then but temporarily, when outraged residents have the temerity to complain to the authorities supposed to be in control.

It is not surprising, therefore, that municipal engineers and the public have a vague feeling that street sprinkling as a method of dust suppression has proved more or less unsatisfactory and inefficient; but, before condemning it and advocating a substitute, it should have a fair trial under the same conditions of intelligent use as its newer competitors.

Mr. Whinery.

The cost of efficient street sprinkling is difficult to determine satisfactorily from the data available. Where referred to at all in municipal reports, the necessary conditions to enable one to deduce reliable figures of unit cost are usually wanting. These reports indicate that the cost, exclusive of water, ranges from less than 1 cent to 3½ cents per sq. yd. for the watering season, which usually extends from May 1st to October 15th. The number of square yards sprinkled once is seldom given, and as the number of times sprinkled per day or per season varies greatly in different cities, and on different streets in the same city, and as there are other varying conditions to be taken into account, it is not possible to get at the basic unit cost from the available records. It is practicable, of course, to estimate the theoretical cost very closely, but such estimates are less satisfactory than actual data from experience.

The statistics available seem to indicate that, exclusive of the cost of the water, the average cost per square yard per season is about 2 cents. The quantity of water reported as used is as variable as the cost of applying it. The figures vary from 20 to 70 gal. per sq. yd. per season. Probably 45 gal. is a fair average, and, at $90 per 1 000 000 gal., the cost for water per square yard per season would be about $\frac{4}{10}$ cent, making the average total cost less than 2½ cents per sq. yd. per season. In Boston, where the accounts are now kept with care, the quantity of water used in 1909 was less than 25 gal., and the total cost 2.1 cents per sq. yd. per season. In the suburban City of East Orange, for the same year, these figures were, respectively, 58 gal. and 2.82 cents, including the cost of water.

If it be asserted that street sprinkling as it is generally conducted has not proved satisfactory, it may be replied that it has seldom, if ever, had a fair trial. If the surface of the street were first cleaned, as is the practice before oiling, the objection that water makes the street muddy and sloppy would be removed. If the specifications required that the street surface be kept constantly moist, regardless of whether one daily sprinkling or five were necessary, there would be no trouble from dust. Where the road material is thus kept properly moistened, the wear of the pavement from travel is not greater than where light oil is used, and it is conceded that, as long as the road material is kept well moistened, the peculiar form of disintegration called raveling does not occur.

As to cost, there is good reason to believe that with careful, intelligent, and economical management, including the restriction of the quantity of water used to that actually necessary, it would not much exceed the average cost of the work as now unsatisfactorily done. Ordinarily, at least, it should not exceed 3 cents per sq. yd. per season, and this figure may be safely used for comparison with other methods.

Calcium chloride has not been used very extensively in the United

States, and, if the speaker is correctly informed, this process is now practiced on a large scale in only one city—Boston, Mass.—where its use has been developed with intelligence, through several years. There, the speaker believes, it is considered fairly successful and satisfactory. According to the Report of the Superintendent of Streets for the season of 1909, 408 511 sq. yd. of roads and streets were cared for during the season by this process, at an average cost of about $4\frac{1}{7}$ cents per sq. yd., or nearly double the average cost of the total sprinkling in the city. This comparison, however, may not be entirely just, because the treatment was used in only two districts—West Roxbury and Dorchester—and, as the report gives the separate cost of sprinkling in these districts, it will probably be fairer to compare the work in them alone, where the average cost of sprinkling was somewhat less than $2\frac{1}{2}$ cents per sq. yd. for the season. The areas treated by the "calcide" process and by water were approximately the same. Mr. Whinery.

It is reported that the average length of time between treatments was 27 days. Experience in other places seems to indicate that more frequent treatments are necessary, in order to keep the streets satisfactorily free from dust. To some extent, the permissible period between treatments depends on the humidity of the air, and, therefore, one would expect that in seacoast cities a single treatment would remain effective longer than in inland cities. Where a long period elapses between treatments, it is natural to expect that there are many days when the humidity is very low, and the streets become unpleasantly dusty. Observations show this to be the fact. During days of such low humidity a satisfactory condition of freedom from dust can be secured only by resorting to sprinkling with water, in order to supply the deliquescent salt with the necessary moisture.

On the whole, there seems to be nothing in the experience of American cities to warrant a preference for this process, since it appears to be less effective and more expensive than sprinkling.

The use of mineral oils for allaying dust is now quite popular with city and road officials, and with the public. The speaker's remarks will be confined to the consideration of the lighter mineral oils, applied primarily for suppressing dust. That the proper application of light oils will suppress dust effectually for some length of time is undisputed; but, is the use of this material more satisfactory, effective, or economical than sprinkling with water? To answer this question positively requires more complete and accurate data than are yet available. Judging from such facts as are now obtainable, however, the speaker believes that it may be safely answered in the negative. To discuss the matter with any degree of fullness would require more time than is now allowed.

That the oil, for a few days after its application, is a nuisance on the streets and results in no little damage to personal property will be

Mr. Whinery.

conceded. The length of time for which a single application will be effective depends, of course, on various conditions, which, unfortunately, are not often given clearly in the records.

In these reports the impression is often conveyed that the oil treatment is effective for the whole season, while, in fact, it is necessary to resort to the use of water for a part of the time in order to lay the dust. In practice it is often found advisable not to apply the oil until the weather becomes settled and the street well dried out, and, where only one application is made, the street frequently becomes very dusty before the end of the season. In such cases sprinkling is often resorted to for a part of the season. For instance, a suburban street, on which the writer lives, was treated with a coat of oil on May 7th. By July 1st the street had become very dusty, and sprinkling with water was resumed until July 20th; then a second coat of oil was applied, which kept the street in good condition until about October 1st. The total cost of sprinkling with water on this street for the season was 1.13 cents per sq. yd. In many such cases the fact that sprinkling had to be used for a part of the season is not mentioned.

In so far as the oiling tends to make the surface of the street impervious to water, it promotes raveling, due to the absence of moisture. The street referred to above was thus affected toward the end of the season, as there was little rain, whereas, during the previous season, which was also dry, the street was sprinkled with water only, and no ravelling appeared. During the previous year the contract cost of sprinkling for the season was less than $400, including the cost of water, while the cost of oiling in the present year was about $600, exclusive of the supplemental sprinkling. The oil was not more effective than water in preventing dust; it was clearly deleterious to the macadam, and was 50% more expensive. It is to be remarked, also, that in this case the cost of the oiling was unusually low. If the extra cost of oiling had been incurred in securing a superior job of sprinkling with water, there can be no reasonable doubt that the condition of the street would have been decidedly more satisfactory during the season.

One objection to oiled streets (observed in the case previously mentioned) seems to have attracted little attention, but, nevertheless, it is worth noting. During the period when the effectiveness of the oil is declining and the street is becoming semi-dusty, high winds pick up the more or less oily particles of dust and deposit them in the houses or on the clothes of passers-by. This oily dust has a great capacity for soiling articles on which it settles, and it is difficult to remove it by ordinary means.

Other methods of allaying dust on streets and roads have been proposed and tried, but, as they have not proved acceptable or economical, they need not be referred to in a brief discussion like this.

Mr. Whinery.

Summing up the conclusions, it may be asserted confidently that no method of suppressing dust, thus far discovered, is more effective and economical than sprinkling with water. Why not, therefore, drop the fads which are being exploited before the public, and turn attention to developing and perfecting this old method, applying to it the same study, care, and efficient management that are devoted to other lines of municipal work.

The speaker believes that there is no practical or mechanical difficulty in developing a power sprinkler which will have double the water and speed capacity of the old sprinkling wagon, and which, at one passage along a street 50 ft. wide, will sprinkle the whole surface effectively from curb to curb. Proper specifications and rigid supervision would accomplish a revolution in the work, both from the standpoints of efficiency and economy.

In a great majority of cases, the common objection that sprinkling the country highways is impracticable because of the frequent absence of a water supply, may be overcome by temporarily installing suitable power pumps at streams and ponds. Where this is not practicable, high-speed power sprinklers would make it possible to obtain the water from comparatively distant sources without increasing the cost of the work materially. With proper watering, the durability of country roads during dry seasons would be increased so much that the sprinkling might almost result in a profit.

What seems to be most needed, for a satisfactory consideration of this subject of dust suppression, is the application of economic and business principles to the several methods, and the collection and dissemination of full and accurate data relating to each. Until such principles are applied, and such data are collected, the indefinite and unprofitable discussion, so common at the present time, will continue.

Mr. Wickes.

J. L. Wickes, Assoc. M. Am. Soc. C. E.—In Baltimore, Md., after experimenting for several years with both light and heavy oils, it was finally decided that, from the standpoint alone of a dust-layer, an emulsion was efficient and much more economical than any of the heavy oils. Accordingly, during the season of 1910, nothing else was used.

The emulsion was prepared under the specifications of the Street Cleaning Department. The first three carloads of oil ordered in the spring were shipped in iron drums, each containing about 110 gal. These drums were distributed by teams to the various macadam streets. All subsequent quantities of oil were delivered in tank cars, and the drums were used for unloading the cars and for storage purposes. There was quite an appreciable reduction in the freight charges by this method, and it is probable that considerable saving was effected.

The method of removing the oil from the tank cars and filling the drums was as follows: The yard where the filling was done had a railroad siding which was high enough to allow the oil to flow by gravity through a 4-in. pipe attached to the bottom of the car and empty into

Mr. Wickes.

the drums. What is known as a tank-car coupling was used on the bottom, and at the coupling the size of the outlet pipe was reduced from 5 to 4 in. Below this there was a short 4-in. nipple, with a 4-in. stop, and a 4-in. bolted flange union. This stop and union were used when the railroad company had to shift the cars, so that the flow of oil could be stopped and the car quickly disconnected. From this union, a 4-in. wrought-iron pipe was laid for a distance of 30 ft., with a 6-in. fall so that the oil could flow freely. At the end of the 4-in. pipe there was a T or cross, and from this there were two 2-in. wrought-iron pipes with stop-cocks, so that two drums could be filled at the same time.

From time to time some difficulty was caused by lumps of asphalt clogging the pipe and impeding the flow of oil; this, however, occurred only in chilly weather. At such times the asphalt base in the oil congealed and formed in large lumps, and it was then found necessary to stir the oil in the car, in order to make it flow freely into the drums. The outlets to these two 2-in. pipes were high enough to permit the drums to be rolled under them, and a large funnel was sometimes used in filling.

There was great difficulty at times in removing the oil from the drums with the siphons, because they became clogged, and there was not sufficient pressure on the plugs to lift the oil from the drums. This was noticed particularly at the low-pressure plugs; at the high-pressure plugs there was no difficulty.

The first application was made with 20% of oil to 80% of water. In the second and subsequent applications the percentages of oil varied from 10 to 5, according to the weather conditions and the amount of traffic on the road. It was found necessary to treat some of the heavy-traffic roads once a week; some of the lighter traveled thoroughfares were treated only once in 2 or 3 weeks, which was found to be sufficient. The length of road treated aggregated about 20 miles, with an area of 441 263 sq. yd., at a total cost of $9 085.63; of which amount, $5 583.92 was for oil and $3 501.71 for distribution and application. These two amounts include all the expenditure for this work, with the exception of the water used and supervision. The entire supervision, however, was done by the speaker in the course of the discharge of other duties, so that no additional expense was incurred on this account. Hence the cost per square yard was a trifle more than 2 cents ($0.0206), which is less than one-half the cost of applying any of the oils used in this work.

The fact that not a single complaint was received relative to dust on any of the roads treated as above described is good and sufficient evidence that the work was done well and efficiently.

For the benefit of those who are contemplating the use of an emulsion instead of straight oil for the treatment of roads, a synopsis of the specifications and report of Messrs. Penniman and Browne, the chemists who made the tests of the material, is given:

Specifications for the Compound.

Mr. Wickes.

The compound must mix with water to form an emulsion so perfect that it can be applied to roads by an ordinary sprinkling wagon without noticeable separation or clogging of the jets. It must be free from objectionable odor, and must consist of at least 75% of asphaltic oil or base.

This asphaltic base must have the following characteristics:

(1) When 20 grammes of the material are heated for 7 hours in a dish, 2½ in. in diameter, to 205° cent., the weight of the residue shall be at least 65 per cent.

(2) When 20 grammes are heated for 21 hours in a flat-bottomed dish, 2½ in. in diameter, at a temperature of 105° cent., the loss shall not exceed 5 per cent.

(3) The percentage of "free carbon" shall not exceed 3 per cent.

(4) The percentage of paraffin shall not exceed 1½ per cent.

(5) The consistency of the base shall be such that 50 cu. cm. shall flow through an Engler instrument in not less than 185 sec. nor more than 300 sec., at 100° cent.

(6) The consistency shall be such that a Lunge tar tester will sink to the mark, 1.4, in not less than 30 sec. nor more than 45 sec. at 25° cent.

Specifications for the Asphaltic Oil.

These are the same as indicated above, except that the base itself is specified and the following clause is added:

(7) It must form a proper emulsion, using the method of the Street Cleaning Department of Baltimore City.

Method of Making Emulsion Oil.

Dissolve 5% of common resin soap in 30% of water and bring to a temperature of 50° cent. Put this soap solution in a churn and add 65% of oil at the same temperature, churn for 20 min. and run into barrels or tanks through a cooler which reduces its temperature to 21° cent. or less. A portion of the compound prepared in this way must remain unchanged for 48 hours at a temperature of 38° cent. When 20% of the compound is mixed with 80% of water at 32° cent. the emulsion formed must remain without separation for 3 hours.

Theory of the Foregoing Specifications.

The base is specified so that it is sufficiently liquid to be broken up by the paddles of the churn at 50° cent., but, on being cooled to 25° cent., the small particles do not run together, being covered with a film of soapy water. The temperature of 38° cent. is also inserted as an additional practical safeguard to prevent the concentrated emulsion from breaking down before use. The dilution test is introduced for the same reason. The manufacturer of the base is thus compelled to

Mr. Wickes. test his product according to the foregoing method of procedure, and shipments are rejected if they are not suitable.

The churning arrangement is not very important, provided the action is that of beating and not merely stirring. A butter churn is efficient, but rather expensive in first cost and power; several paddle wheels attached to the vertical shaft have been found to work efficiently, and are cheaper. A churn having a capacity of from 300 to 500 gal. is a convenient size. The necessary heating tanks are evidently matters of individual requirement, and require no description in detail. The process is most likely to break down through lack of efficient cooling between the churn and the storage tanks. The cooler should be of sufficient capacity to chill the material, and, of course, must be well supplied with water and have proper arrangements for heating and storage. Two men should have no difficulty in making from 8 to 10 batches per day.

Mr. Poore. H. C. Poore, Jun. Am. Soc. C. E.—The speaker's experience has been mainly with bituminous construction, but, during the past season, 300 000 sq. yd. of macadam and gravel surface have been treated under his supervision with a light refined coal-tar, for the purpose of laying the dust and incidentally preserving the road surface. A perfect dust layer should also prevent the road surface from "potting out" or raveling.

The general method of application was as follows: After the surface had been thoroughly swept, thin refined tar ($\frac{1}{2}$ gal. per sq. yd.) was applied by a pressure-tank wagon, sprinkler wagon, or by some form of hand-dipper. After a few hours the surface was covered with grit. Both pea stone and fine gravel were used, and 1 cu. yd. of material usually sufficed to cover 120 sq. yd. of surface. Where the entire road or only a portion could be closed and guarded for 24 hours after spreading, the coating of grit was not always necessary. The average cost of this work in Massachusetts was from 3 to 5 cents per sq. yd., including all materials and labor.

With this treatment, a macadam or gravel road in fair condition will become hard and free from loose grit in the course of about 10 days. The surface then remains in condition for the season, and, with a re-treatment of $\frac{1}{4}$ gal. per sq. yd. in the fall or following spring, will be in good condition for a second year. The second treatment may be applied at a greatly reduced cost.

In various localities a thin asphaltic oil is laid in practically the same way on some streets, while on others, where it is desired to have a clean and hard surface which will remain in good condition through rainy and variable fall weather, refined coal-tar is preferred. Since a tarred surface is hard, and does not assimilate all foreign matter dropping on it, but easily allows the street to be cleaned by hand or by

Nature, tar materials usually remain intact during the entire season, and in some cases throughout the winter. Mr. Poore.

In one large city in Maine, where refined coal-tar was used for dust-laying on a large area, but where a larger quantity per square yard was applied than is customary, a dustless surface has been in existence for 2½ years, with no maintenance cost whatever. This is not to be considered as a heavy film treatment, as the surface was mosaic, and the tar was absorbed by the road materials.

A very efficient and economical way of applying a thin tar for dust prevention is by using 500-gal. tank wagons, fitted with auxiliary pressure tanks for blowing out the material through suitable nozzles under pressure. The use of a hose with a nozzle is a very practical method of distributing all light materials, and when they are delivered by pressure, the spreading of a ¼-gal. coat to the square yard is easily accomplished. Two applications of this quantity of material during the first year, followed by one coat during the second year, will suffice to keep the surface of a macadam road in first-class condition.

Charles W. Ross, Assoc. Am. Soc. C. E. (by letter).—Of the palliatives used in Newton, Dustoline and various oil emulsions are of the greatest interest. Dustoline was used on about 40 000 sq. yd. of one of the main thoroughfares, and the cost for the season was less than 3 cents per sq. yd. of roadway. Mr. Ross.

The principal emulsions used on the streets included in the street-sprinkling system are "Terracolio" or "Headley's Dust Preventive No. 1," "Speare's Road Binder," and "Standard Emulsifying Road Oil." These were mixed in the proportions and rate of 50 gal. of the emulsion to 500 gal. of water, and sprinkled on the roadway from an ordinary water cart. Most of these materials last from 1 to 3 weeks, according to the weather.

Hubert K. Bishop, M. Am. Soc. C. E.—In New York State some roads, under the speaker's direction, have been treated with calcium chloride in the dry or granular form. The material as furnished was packed in steel drums, and cost $13 per ton, plus the freight rate. It was applied to quite a number of miles of road with fairly good success. Three applications were usually made, consisting of about 1 lb. per sq. yd. for the first, and from ½ to ¾ lb. for the second and third applications. The material was put on the road by the patrolmen, with the aid of a very small machine which spread it uniformly; it was then allowed to lie on the road until it was fully absorbed. It keeps the road moist, dustless, and preserves a sort of cushion with the screenings, which protects the road from wear. It could not be used on every road, but on a great many in New York State it was very successful. The cost runs from $125 to $150 per mile, the cost for labor being very low. Mr. Bishop.

Mr. Grabill. L. R. GRABILL,* ESQ.—In Washington, D. C., calcium chloride was used on macadamized residence streets last season, and was found to be quite satisfactory. Sometimes more than one application for each 3 weeks was made, to a great extent depending on whether the weather was damp or extremely dry.

There is one point that a great many users, and perhaps prospective users, of calcium chloride are apt to lose sight of, namely, that the satisfaction obtained from the use of any compound which derives its moisture from the atmosphere will necessarily depend on the condition of the atmosphere, and such a compound is much more likely to be satisfactory in a climate where there is much hygroscopic moisture than in an inland city. The speaker thinks this is probably the reason for the failure of this process, as reported, in some inland cities, whereas it has given satisfaction in seacoast cities.

The cost of the work in Washington for the latter half of the season was slightly in excess of the cost of water, but the streets were kept in better condition continually than they would have been if sprinkled by the ordinary method.

Mr. Meeker. R. A. MEEKER, ESQ.†—Water has been used on some New Jersey roads, but in a peculiar manner. In the summer the water was applied after sundown, not to lay the dust, but to retain the cementing properties of the stone. A stone road will very often attain a temperature of 110° Fahr. and water sprinkled on it during the day simply evaporates, without doing any good. To overcome this objection some roads have been sprinkled at night, with very good results. It is true they are somewhat dusty after midday, but they do not disintegrate or break to pieces, and the fine material is not blown off. A road which is well constructed of water-bound macadam and then treated with the lighter oils, or even with the lighter tars, yields fairly good results if all ruts and depressions are removed before applying anything in the nature of a dust palliative.

Many of the failures resulting from the use of dust layers are due to the bad condition of the surface at the time the liquid was applied.

Mr. Whinery. S. WHINERY, M. AM. SOC. C. E. (by letter).—In the introductory discussion the writer omitted details relating to the methods of dust prevention, because he understood that he was limited to ten minutes. Among these details was the use of emulsions of oil and water for dust laying, and Mr. Wickes' interesting account of the use of such an emulsion in Baltimore, Md., is, therefore, a welcome addition to the discussion.

From the facts available and from experience the writer is inclined to believe that emulsions of oil will be found, on the whole,

* Engineer Dept., District of Columbia, Washington, D. C.

† State Road Supervisor of New Jersey.

equally satisfactory and more economical than oil alone, when dust prevention is the main object. Mr. Whinery.

In these emulsions the oil is applied in a finely divided state, and can be more evenly distributed over the surface, and more effectually incorporated with the fine material on the street, than when applied alone in bulk. It also tends to prevent the rapid evaporation of the water, and thus prolongs the effectiveness of the application. The oil, so applied, has also some value as a binding material to consolidate the fine material on the surface of the street. It is still a question, however, whether the increased effectiveness of the emulsion is sufficient to compensate for the usually greater cost of treatment.

The details of the cost in Baltimore are not sufficiently full to allow one to compute the actual unit cost. It appears that the oil itself costs at the rate of about 1¼ cents per sq. yd., and that the whole cost, exclusive of the value of the water used, was a fraction more than 2 cents per sq. yd. cared for during the season. Even if the cost of the water were added and a reasonable allowance for superintendence made, the total unit cost must have been very reasonable, and it is stated that the service was satisfactory.

In further experiments along this line the writer would suggest the possibility of emulsifying the oil and water without the use of soap, by thorough mechanical agitation alone. Experiments made by him a few years ago, for another purpose, indicated that this is probably practicable.

(5) SURFACE TREATMENT WITH TARS, HEAVY OILS, ETC.

By Messrs. Charles W. Ross, W. W. Crosby, Prevost Hubbard, Arthur H. Blanchard, Frank J. Eppele, John R. Rablin, R. K. Compton, William H. Connell, Arthur W. Dean, Fred. E. Ellis, Harold Parker, Michael Driscoll, Philip P. Sharples, Hubert K. Bishop, R. A. Meeker, and James Owen.

Mr. Ross. Charles W. Ross, Assoc. Am. Soc. C. E. (by letter).—The last few years have witnessed an increased use of various materials for the treatment of roads, with a view to their preservation, as well as the prevention of dust. The methods have been of two classes, namely, by improved processes of road construction, and by surface treatments.

With the advent of the automobile, new conditions confronted road engineers, and they were forced to decide at once whether they would use materials for simply suppressing dust, such as oils or emulsions which act partly as binders, or bituminous liquids which would prevent wear and disintegration by forming a water-tight wearing coat.

The continuous and fast motor traffic to which roads are subjected denudes them of the fine cementing surface material. The effect of such traffic on stone roads has been the subject of much study by highway engineers in nearly every civilized country, and many theories have been advanced to account for the injury to the roads. It has been generally agreed that the suction of the pneumatic tire on the surface is the main cause of the trouble; but in a recent address, Logan Waller Page, M. Am. Soc. C. E., advanced the theory that the great tractive force, or longitudinal shear exerted by the driving wheels of motor cars is the main cause of the injury. This is doubtless due largely to irregularities on the surface of the road. The slip, of course, increases the quantity of finely-divided surface material which is thrown into the air, and, as the amount of damage increases in proportion to the irregularities of the road and the speed of the car, the effect will be greatly reduced if the road has a smooth surface.

The prevention of dust offers two distinct problems: In the first place, the question naturally arises as to the methods to be adopted in the construction of new roads in order to bring about the desired properties of freedom from dust and low cost of maintenance. The second problem is that of the treatment of roads already in existence, to prevent wear and disintegration.

There are two general methods by which such roads are treated to remedy the dust nuisance, although it is difficult to draw a definite line between them. The first is by sprinkling the surface with any of

the emulsions, and the second is by treating the road with a binding material so that only a minimum amount of dust will be formed. The first method of suppressing the dust is of necessity a temporary expedient. Water, salt solutions, light oils, emulsions, etc., come properly in this class. In the second method, it is obvious that the essential properties in a preservative are its binding power and durability. For this reason, and by way of distinction from temporary treatments, materials for this purpose can be properly called permanent binders. Such materials as tar and heavy asphaltic oils are included in this class. It must be pointed out, however, that the word "permanent" is only used in a relative sense, and that binders which on application will last at least one whole season, are included under this head. Mr. Ross.

The principal agencies which tend to cause deterioration on tarred and oiled roads are frost, heavy rains, and decaying organic matter on the surface of the road. In the writer's opinion, this last mentioned cause is more destructive than either of the others, and, for this reason, after a street is treated, it should be swept, by hand sweepers or otherwise, often enough to keep it clean. As far as can be judged from experiments, one kind of road resists the action of these agencies as well as another.

Oil, as a rule, penetrates better than tar, but its value is in the asphalt base, and, before the maximum binding power is reached, the more volatile constituents of the oil must evaporate, which is a slow process. A properly treated road should resemble sheet-asphalt pavement, although it is of a more resilient character. The surface material is all bonded together by the binding material, forming a smooth, firm surface which can be cleaned in the same manner as a sheet-asphalt pavement.

Probably as early as 1894 crude oil was used for road treatment in Santa Barbara, Cal. In 1898 six miles of road were treated in Los Angeles County, simply with the idea of laying the dust. The experiment met with astonishing success, and the practice spread rapidly. The oil not only stopped the dust nuisance, but bound together the particles of the road surface, forming a tough layer resembling asphalt pavement. Since California made these first experiments, oil has been used extensively in many parts of the country. It has been found that the quality of the oil used is of the utmost importance.

Too much stress cannot be laid on the fact that the street must be thoroughly clean before any of the material is applied. It is not necessarily advisable to have the road surface absolutely dry, practice having shown that the oil has a tendency to adhere to the dry dust on the road. This can be prevented by sprinkling the surface with water two or three hours before the work is to be done. The oil then penetrates, and prevents the pulling up of the oil and sand.

Mr. Ross.

The writer has tried most of the asphalt preparations in use at the present time for preserving the road and rendering it dustless, and has classified these in three grades, as follows:

Grade A.—This is an asphaltic oil, sufficiently fluid to be applied to the roadway at atmospheric temperature, and is used on the surface of a finished road. It will effectually bind the material in the road, render it water-proof, and protect it from wear of traffic of all kinds. Automobile traffic tends to iron it out and improve it, rather than destroy it. It renders the surface resilient and elastic, and provides an agreeable tread, both for horses and vehicles.

Grade B.—This is a heavier material than Grade A. It is used for the building of new roads, or for resurfacing or reconstructing old ones, and is applied hot to the second course of stone, as set forth in the specifications under what engineers define as the "Penetration Method." It makes a solid bituminous road, extending from 2 to 3 in. below the surface, and will last and remain dustless for several years.

Grade C.—This is a semi-solid bituminous product, and is used in building or resurfacing roads by the "Mixing Process." It produces a hard, smooth, dustless roadway, absolutely water-proof, and cemented so solidly that a pickaxe cannot displace or disarrange it without great effort and labor.

At West Newton, Mass., since adopting the new methods of work, large quantities of Asphaltic Oils, Tarvia, Tarite Asphalt, and Standard Macadam Binder have been received. The materials come in tank-cars which are heated and unloaded in the following manner: The car is first connected with a small steam boiler, and heated to a temperature of about 100° Fahr., thus rendering the material thin enough to flow through the hose at the bottom of the car, which rests on a trestle about 10 ft. above the surface of the ground.

When adaptable to the work, a Studebaker tank-wagon is used in applying these materials to the roadway. This wagon consists of a steel tank, holding about 550 gal., with forty perpendicular tubes, 2 in. in diameter, passing through it. Under this tank two large burners are supplied with kerosene oil forced in by pressure. The fire-box, extending the whole length of the tank, gives an even heat. The material is applied through a sprayer or spreader which is perforated so that the oil trickles down on the roadway in very fine streams, covering it thoroughly in one application. In the writer's opinion, this is a very satisfactory way of applying any heavy asphalt, as the material can be readily heated, and can be applied in large or small quantities, the flow being controlled by valves which can be regulated so that they will spread as small a quantity as $\frac{1}{8}$ gal. per sq. yd. In applying these materials it is better to close half the roadway at a time, leaving the other half open to traffic.

Asphaltoilene was used in 1907 on two roadways in Newton, a

surface of about 16 822 sq. yd. being treated at the manufacturers' contract price, 6 cents per sq. yd. At present these roads are in very good condition. Mr. Ross.

Several macadam-surfaced streets, having varying grades up to a maximum of 9%, and subjected to heavy horse-drawn and automobile traffic, were selected for the liquid-asphalt treatment. The method used was as follows: A quantity of sand was heated to a temperature of about 200° Fahr., dumped in a pile, and leveled. The asphalt was poured over the hot sand in the proportion of 1 gal. to each cubic foot of sand, and then the whole mass was turned with shovels or mixed in a concrete mixer (the latter being preferable on account of cost). This work was done at the pit. The mixture was teamed to the work and spread on the roadway to a depth of less than ¼ in., being raked even with 14-tooth wooden rakes. Rolling was not considered necessary, and the street was kept open for traffic at all times. The cost of this treatment was about 3 cents per sq. yd. It has the advantage of leveling and building up the surface of the road, each new application providing a new wearing surface. This work has remained in perfect condition without further expense since the summer of 1909.

Some cities have had very good results by using the following process for patching. Gravel from an ordinary gravel pit is run through a ½-in. screen, using a little more than ½ gal. of the Asphaltoilene, or any of the heavy oils, to a cubic foot of gravel. It is better to heat the oil or the gravel, unless the work is done in the summer, in which case there is no need of heating. A quantity of material can be prepared at the pit and kept ready for use at short notice. The writer has found that the best time to apply this treatment is after a heavy rain, as all the depressions in the roadway show little pools of water, and if these are filled with the mixture and the surface is raked, it saves the expense of resurfacing the entire street.

The idea of applying tar to the surface of broken-stone roads probably originated with an Italian physician, Doctor Guglielminetti, the method being first applied to the streets of Monaco. The first authentic mention of a practical application of coal-tar to the surface of a completed macadam road was in 1880, when a street near Bordeaux was given a surface treatment. The results were unsuccessful, but 6 years later a similar experiment was made at Melbourne, Australia, with so much better results that the practice was continued. More care was taken in this experiment, the road surface being first cleaned, and after the tar was applied the road was given a covering of fine crushed stone. The results showed that tar had considerable value as a dust-layer by cementing the surface of the road together, thereby preventing its deterioration. The wear on the road was diminished, and consequently the cost of maintenance was very materially reduced. Since that time, other experiments have been tried, in France, Eng-

Mr. Ross. land, and the United States. The use of tar on road surfaces in the United States has been confined largely to the East and North, except in a few instances where the park authorities in the larger cities have made some experiments.

One of the first experiments of any importance in the United States was made at Jackson, Tenn., in the summer of 1905. The United States Office of Public Roads, in co-operation with Mr. Samuel C. Lancaster, City Engineer of Jackson, made a series of careful experiments to determine the value of coal-tar treatments on broken-stone roads. These experiments created such widespread interest that similar ones were made in different sections of the country.

The State of Massachusetts was among the first to use Tarvia, a number of cities in the vicinity of Boston, as well as the Metropolitan Park Commission, and the State Highway Commission, treating several miles of roadway. At about this time Rhode Island and New York were using Tarvia and other tar preparations on their State roads and streets with good results. The foregoing experiments show conclusively that a specially prepared tar has given better results than any of the crude tars in use up to the present time.

In Newton the first Tarvia was used on a section of the boulevard, in September, 1906. The roadbed was first repaired and shaped up. The area covered was 3 500 sq. yd., and the cost was 14 cents per sq. yd. This section has not received any repairs up to the present time, and is apparently in first-class condition in every way.

In 1907 a section of a road, the macadam surface of which had become badly worn, was treated as follows: It was spiked up with the roller and covered with a light grouting of 1:2 Portland cement mortar, which was spread evenly over the entire surface, and penetrated into the stone in such a way as to bind the whole material together. It was then smoothed off with large street-sweepers' brooms. The road was closed for about 4 days, giving the cement time to set. After this it was opened to traffic and used for about a year. Then the surface was swept with an ordinary horse-sweeper, and covered with a thin coating of Tarvia A, heated to a temperature of about 170° Fahr., which adhered readily to the cement, giving a smooth and satisfactory surface. This still remains in first-class condition, after a period of about 3½ years. On a small portion of the same street a second coating of cement grout was applied and covered with Tarvia A. This experiment was for the purpose of ascertaining whether the materials would adhere and form a satisfactory surface. It has proved to be all that was expected.

In recommending the use of the different tar treatments, it is well to bear in mind that they are better adapted to macadam than to gravel surfaces, because the surface is likely to become broken unless the foundation is solid enough to carry the load, and, once broken, deterio-

ration is very rapid. The asphalt oils, however, can be applied to either. They stand the winter weather without breaking, and in the spring are as smooth as when built. Mr. Ross.

Within the past two or three years, or since the automobile came into general use, many engineers have almost come to the point of deciding against the construction of macadam roads. While this may be true as relates to water-bound macadam, yet, in view of the small cost of building and maintenance, as compared with the more expensive forms of laid pavements, it is questionable if anything better than macadam has yet been discovered for modern traffic in towns and small cities, provided it is bound or surfaced with bituminous preparations. Experience has shown that if a macadam road is properly treated when first built or extensively repaired with any of the foregoing bituminous binders, the results will be satisfactory; and, considering the difference in cost between the old form of water-bound macadam and the new method of using bitumen, the amount is very slight compared with the advantages of a roadway which is water-proof, dustless, easily cleaned and repaired, and will endure both horse-drawn and mechanically propelled vehicles. The automobile is not so much to be dreaded as was at first supposed. A road properly treated will stand the traffic of automobiles and horse-drawn vehicles and wear for as many years as it would without mechanical traction.

The progress which has been made in the past 10 years is very gratifying. The time has now arrived when engineers must require as effective construction as possible, which practice is always the cheapest in the long run.

W. W. Crosby, M. Am. Soc. C. E.—During 1910, under the direction of the speaker, about 50 miles of road in Maryland were treated with various tars and heavy oils, most of this work being on newly finished water-bound macadam. For the sake of conciseness, the information concerning this construction is submitted in Table 1. Mr. Crosby.

For the road between Baltimore and Washington, portions of which had been improved with water-bound macadam in previous years, the information is given concisely in Table 2.

The traffic over the roads mentioned in Table 1 has been counted and is given in Table 3.

The traffic over the sections of the Baltimore-Washington Road has been counted, and is given in Table 4.

In September, 1908, a 3 year-old stretch of water-bound, trap-rock macadam, in fair condition, 12 ft. wide and 538 ft. long, was swept thoroughly, and coated with refined water-gas tar to the extent of 1.08 gal. per sq. yd. A coating of stone chips was then applied in sufficient quantity to take up any excess of tar which did not penetrate the road itself in about 5 hours. The chips were then well rolled. The cost was as follows:

Mr. Crosby.

Pitch (at 6 cents per gal. delivered)	6.6	cents per sq. yd.
Stone chips (at $1.75 per ton delivered)	2.3	" " " "
Labor and incidental expenses	3.8	" " " "
Total	12.6	cents per sq. yd.

This section has been, and is now, in an entirely satisfactory condition; it has not needed any attention or expense since construction. The traffic on this road, as counted recently, was:

Single vehicles	66	Average traffic per day of 12 hours.
Double vehicles	9	
3- (or more) horse vehicles	8	
Motor cycles	3	
Runabouts	3	
Touring cars	41	

It is impossible to give the ultimate cost of the above work now, as the data when re-treatment will be necessary are unknown and at present not even in sight.

In addition to the foregoing, a material, which might perhaps be included as a "Heavy Oil," for the purpose of reporting on it under this topic, has been tried.

In November, 1909, a section of Park Heights Avenue, about 6 miles from the City Limits, was treated by simply spraying "Glutrin" from an ordinary sprinkling cart on the surface. The quantity used was 1 600 gal. on 2 300 sq. yd. of water-bound limestone macadam which was built in 1908, and in November, 1909, was still in very good condition. The limestone was of the white variety, rather soft, though highly cemetitious. The "Glutrin" was spread about 1 ft. beyond each edge of the 12-ft. wide macadam, as the shoulder material was ordinarily of poor quality. The traffic over this section of road had been fairly light, and, until the fall of 1910, the "Glutrin" treatment was visible and seemed to have had some good effect. The road surface of the treated section was less friable and less dusty than the untreated macadam of the same road immediately beyond. The cost of this "Glutrin" treatment in 1909 was about 10 cents per sq. yd. During 1910 a branch road was opened into Park Heights Avenue at about the middle of the "Glutrin"-treated section, and hence the traffic over a part of it greatly increased.

The following is a recent count below the branch road referred to:

One-horse	12	Average traffic per day of 12 hours.
Two-horse	1	
Four-horse	5	
Six-horse	13	
6- or 7-seat cars	1	
Omnibuses or drays	1	

Mr. Crosby.

TABLE 1.

Road, reference No., and when done.	Length, in miles.	Width, in feet.	Kind of material in macadam.	Material used for surface treatment, and quantity per square yard.	Covering used and quantity per square yard.	Total cost of treatment, per square yard, or contract price.
Forestville-Marlboro Contract No. 0130 November, 1910	3.19	14	Limestone.	U. G. I. dust-layer 0.527 gal.	Sand. Pea gravel. 10.83 lb.	$0.049
Greensboro-Denton Contract No. 050 November, 1910	2.59	14	Trap rock. Limestone screenings.	Asphaltoilene. 0.518 gal.	Sand. Pea gravel. 18.11 lb.	0.056
Denton-Federalsburg Contract No. 051 November, 1910	1.15	14	Trap rock. Limestone screenings.	Asphaltoilene. 0.597 gal.	Sand. Gravel. 18.64 lb.	0.066
Hickory Contract No. 0175 October, 1910	2.38	14	Trap rock.	Texaco special. 0.6871 gal.	Roland Park Gravel. 9.40 lb.	0.086
Forest Hill Contract No. 0171 October, 1910	1.93	14	Trap rock.	Texaco special. 0.509 gal.	Roland Park Gravel. 9.49 lb.	0.066
Easton-Wye Mills Contract No. 0110 August, 1910	3.03	14	Limestone.	Asphaltoilene. 0.492 gal.	No covering.	0.05*
Federalsburg-Hurlock Contract No. 052 September, 1910	1.12	14	Limestone.	Standard Oil No. 5. 0.519 gal.	Sand. 7.84 lb.	0.03
Federalsburg-Hurlock Contract No. 070 September, 1910	5.36	14	Limestone.	Standard Oil No. 5. 0.504 gal.	Sand. 11.36 lb.	0.028

* Contract price.

TABLE 2.

Section, and when macadamized.	Length, in miles.	Width, in feet.	Material of macadam.	Cost of repairs to macadam, per square yard.	Kind of material used for surface treatment, and quantity per square yard.	Covering used, and quantity per square yard.	Cost of surface treatment, including covering, per square yard.	Total cost of repairs and treatment, per square yard.
Section No. 9..... 1909	2.76	14	Limestone.	$0.0176	Standard No. 5. 0.497 gal.	Pea gravel. 18.49 lb.	$0.029	$0.047
Section No. 7..... 1909	1.80	14	Limestone. Trap rock.	0.0526	Standard No. 5. 0.442 gal.	Pea gravel. 18.31 lb.	0.036	0.088
Section No. 5..... 1908	2.30	14	Trap rock. Gravel.	0.0496	Standard No. 5. Fairfield Anti-Dust. 0.54 gal.	Pea gravel. 13.26 lb.	0.033	0.083
Section No. 1..... 1906	0.53	12	Trap rock.	0.0165	Standard No. 5. 0.503 gal.	Sand. 12.05 lb.	0.025	0.19
Section No. 2A..... 1909	0.19	16	Trap rock.	0.0145	Standard No. 5. 0.6725 gal.	Sand. 12.30 lb.	0.034	0.048
Section No. 4A..... 1910	0.78	14	Trap rock.	0.0215	Standard No. 5. 0.547 gal.	Sand. 13.33 lb.	0.031	0.056
Section No. 8A..... 1907	1.63	14	Limestone.	0.034	Asphaltoilene. 0.564 gal.	.....	0.05	0.084*
Section No. 8B..... 1909	1.57	14	Limestone.	0.0704	Asphaltoilene. 0.564 gal.	.....	0.05	0.057*
Section No. 12B.... 1909	1.59	14	Limestone.	0.0225	Asphaltoilene. 0.564 gal.	.....	0.05	0.072*
Section No. 12A.... 1908	0.97	14 16 18	Limestone.	0.0321	Asphaltoilene. 0.564 gal.	.....	0.05	0.082*
Section No. 14A.... 1909	0.41	18	Limestone.	0.0156	Asphaltoilene. 0.564 gal.	.....	0.05	0.065*
Section No. 3..... 1908	2.61	14	Trap rock.	0.104	Standard No. 5. 0.457 gal.	Sand and gravel. 26.046 lb.	0.028	0.132

* Pitching done by contract.

Mr. Crosby.

TABLE 3.—AVERAGE TRAFFIC PER DAY OF 11 HOURS.

Mr. Crosby.

Reference number of road.	One-horse.	Two-horse.	Three-horse.	Four-horse.	More than four-horse.	Motor cycles.	Runabouts.	4- or 5-seat cars.	6- or 7-seat cars.	Motor drays.
Forestville-Marlboro. Contract No. 0130.....	27	19	0	0	0	0	0	0	0	0
Greensboro-Denton. Contract No. 050.....	132	32	0	2	0	1	3	4	0	0
Denton-Federalsburg. Contract No. 051.....	213	36	0	2	0	2	7	10	0	0
Hickory. Contract No. 0175.....	43	9	0	3	1	1	0	0	0	0
Easton-Wye Mills. Contract No. 0110.....	197	70	0	0	0	1	4	2	0	0
Federalsburg-Hurlock. Contract No. 052.....	54	10	0	0	0	0	4	4	1	0
Federalsburg-Hurlock. Contract No. 070.....	54	10	0	0	0	0	4	4	1	0

TABLE 4.—AVERAGE TRAFFIC PER DAY OF 10.5 HOURS.

Section number.	One-horse.	Two-horse.	Three-horse.	Four-horse.	Six-horse.	Motor cycles.	Run-abouts.	4- or 6-seat cars.	7-seat cars.	Wagons.
9	115	25	1	0	0	20	3	29	11	1
2-A	151	26	0	1	1	1	1	7	3	1
4-A	71	12	0	0	0	0	2	6	0	0
12-B	38	33	0	2	0	4	3	19	4	0
12-A	233	98	2	2	0	3	6	20	10	0
14-A	267	142	3	4	0	7	4	33	5	0

It was decided to treat the main road below the branch road with bituminous material, which was done. The remainder of this road, to its end (about 10 000 ft.), needed some repairs to the macadam. These were made by water-binding methods, at an average cost of $0.276 per sq. yd., calculated on the basis of the total area of the macadam in the road, as the road was built in 1908 and had received practically no maintenance. Then "Glutrin" was applied to the macadam through an ordinary sprinkling wagon.

The proportions used were 5 gal. of "Glutrin" to 2 gal. of water, or 0.426 gal. of "Glutrin" per sq. yd. of macadam. It was sprinkled about 2 ft. outside the edges of the macadam. The work was completed about December 1st, 1910. The cost was as follows:

Repairing macadam.....	$0.276 per sq. yd.
Treating with "Glutrin"	0.07 " " "
Total cost.....	$0.346 per sq. yd.

The traffic on this section, as counted recently, was:

One-horse	15	Average traffic per day of 12 hours.
Two-horse	4	
Four-horse	2	
Six-horse	5	
Runabouts	2	
4- or 5-seat cars.....	10	
7-seat cars.....	2	

Mr. Hubbard.

PREVOST HUBBARD, ESQ.*—Without doubt, the most vital question to be considered in connection with the surface treatment of macadam roads with heavy oils is: Under what conditions will such treatment prove satisfactory? Apart from the condition of the road surface before treatment, the character of the oil, and the method of application, a most important factor is the traffic to which the road is, or will be, subjected. Considerable attention has recently been paid to the first three, but the last seems to have been overlooked to a great extent, at least, in so far as available data are concerned. It is to this factor—the effect of traffic on macadam roads surfaced with heavy oils—that the speaker wishes to call particular attention.

Granting that a given road in good condition has been treated properly with a good grade of oil, experience indicates that horse-drawn traffic will damage the surface mat of oil and stone screenings to a greater extent than automobile traffic. In fact, the latter, if not excessive, seems in certain ways to be beneficial to such a surface, by keeping it smooth and well compacted. Horse-drawn traffic, on the other hand, tends to cut and roughen the oil mat, and if the ruts and caulk marks are not ironed out promptly by the passage of motor vehicles, the mat is apt to disintegrate and produce an oily, disagreeable mud in wet weather. In general, it may be said that such damage is most pronounced during cold, rainy weather, and that an oil-surfaced road will prove most unsatisfactory at such times. This being true, particular attention should be paid to traffic conditions during bad weather.

In ascertaining from previous experiments under what conditions surface treatment will prove satisfactory, the following facts are most necessary:

(1) The volume of traffic in bad weather;
(2) The proportion of horse-drawn vehicles to motor vehicles during bad weather;
(3) The character of both classes of traffic during bad weather.

A certain instance has come under the speaker's observation: Two macadam roads, in good condition, were treated in the same manner, and with the same oil, as shown by analysis. One of these roads proved to be quite good throughout the year, while the other was far from satisfactory during bad winter weather, at which time an oily mud was formed over the surface to a depth of about 2 in. Eventually, this mud had to be removed. The oil was a heavy semi-asphaltic residuum, barely fluid enough to apply cold in a satisfactory manner. The following are the specifications for this oil:

* Chemist, Office of Public Roads, Washington, D. C.

Specifications for Road Oil. Mr. Hubbard.

This oil is to be used cold in the surface treatment of clean, well-consolidated macadam roads.

(1) The oil shall be a viscous fluid product, free from water, and showing some degree of adhesiveness when rubbed between the fingers.

(2) It shall have a specific gravity of not less than 0.940 at 25° cent.

(3) It shall be soluble in carbon bisulphide, at air temperature, to at least 99.0%, and shall show not more than 0.2% of inorganic matter insoluble.

(4) It shall contain not less than 3.0%, nor more than 10.0%, of bitumen insoluble in 86° Baumé paraffin naphtha at air temperature.

(5) When 240 cu. cm. of the oil are heated in an Engler viscosimeter to 50° cent. and maintained at this temperature for at least 3 min., the first 50 cu. cm. shall flow through the aperture in not less than 10 min., nor more than 20 min.

(6) When 20 grammes of the material are heated for 5 hours in a cylindrical tin dish, approximately 2½ in. in diameter and 1 in. high, at a constant temperature of 163° cent., the loss in weight by volatilization shall not exceed 20 per cent. The residue should be decidedly sticky.

(7) Its fixed carbon shall be not less than 3.5 per cent.

A traffic census of the two roads, taken on various days during bad weather, showed that it was quite uniform at such times. For both, the traffic consisted mainly of automobiles, light carriages, and delivery wagons, no heavy trucking being allowed. The maximum traffic on the road in good condition amounted to 41 vehicles per hour, while on the road in bad condition there were 127 vehicles per hour, giving a proportion of about 1 to 3. This proportion was practically the same for each hour taken. On the road in good condition, 5 automobiles were counted for every 4 horse-drawn vehicles, while on the bad road the proportion of automobiles to horse-drawn vehicles was approximately 1 to 2. In view of the fact that on neither of these roads can the volume of traffic be considered as excessive, it seems probable that the difference in results can be attributed directly to the greater proportion of horse-drawn vehicles to motor vehicles.

Of course, too great reliance should not be placed on a single set of observations, but the following facts would seem to be indicated from these experiments:

(1) With the type of oil used, satisfactory results may be expected when the automobile traffic is in excess of the horse-drawn traffic during bad weather;

(2) With this same oil, unsatisfactory results may be expected when the proportion of horse-drawn vehicles is double that of motor vehicles during bad weather.

In the speaker's opinion, the danger line lies much closer to the point where the proportion of horse-drawn vehicles just exceeds that of

Mr. Hubbard. motor vehicles during bad weather, but this remains to be proved. It would seem reasonable, however, to expect that the number of automobiles must be equal to the number of horse-drawn vehicles in order that the marks on the oil mat made by the latter may be ironed out as fast as formed.

The limited time necessarily allowed for these discussions makes it impossible to enter into as much detail as might be desired, and therefore the speaker will conclude with the suggestion that in future surface treatments, careful records of traffic during bad weather be kept, in order to determine when such treatment can be expected to prove satisfactory. This information should be particularly valuable to those who have occasion to treat park and pleasure drives, as surface treatment would seem to be of the greatest value on roads of that type.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—The extent and the success of the future use of surface treatments in the United States depends on the development of economical and efficient methods for the distribution of bituminous materials, the use of methods and materials adaptable to local conditions, the construction of macadam and gravel surfaces in such a manner as to be suitable for the effective superficial application of bituminous materials, and the adoption of an efficacious system of continuous maintenance, including the installation of proper plant equipment. That superficial treatments are entirely satisfactory from the standpoints of economics and efficiency when the above conditions are fulfilled, the practice of English and French engineers during the past five years clearly demonstrates.

The present seems to be an opportune time to discuss some of the disputed points relative to methods and materials used in connection with surface treatments. Not only is there a decided difference of opinion among American engineers as to the value of various methods and materials, but foreign engineers—especially the English and French—also hold widely divergent opinions with reference to certain details. Among the points which should engage the attention of engineers are the following: The relative advantages of pressure and gravity-flow distributors; the amount of penetration of an ordinary macadam road obtained by the use of a pressure spraying machine; the relative advantages of sand, shingle, gravel, and stone chips as a covering; the feasibility of the omission of the covering; the practicability of drying a damp macadam surface artificially in order to render possible the application of such a bituminous material as tar, thus obviating the unnecessary expense due to waiting for the road to dry out; the relation of the size of stone composing the surface of a macadam road to the economics and efficiency of surface treatments; the slipperiness of surfaces treated superficially with various bituminous materials under varying climatic conditions; the effect of non-skidding devices, particularly chains, steel studs, and rubber blocks, on

various surfaces; and cost data covering a series of annual applications of a given kind of bituminous material on the same road. Mr. Blanchard.

FRANK J. EPPELE, ESQ.* (by letter).—It is not intended to enter into a technical discussion of the relative values of the various methods of surface treatment or even to consider the comparative merits of the numerous compounds used, but to confine the discussion to a review of the work done by Mercer County, New Jersey, in maintaining its system of improved highways. Mr. Eppele.

That work has been surface treatment, and has been confined almost entirely to the application of a bituminous dust layer known as "No. 4 Road Oil," guaranteed to contain from 40 to 45% of petroleum asphalt, and "No. 5 Road Oil," having an asphaltic content of from 50 to 55 per cent. The primary object in using this oil compound was to allay dust, but it was also found that its application added materially to the condition of the surface of the road, and, for the time being, obviated the necessity of making more costly repairs. It was demonstrated that on macadam roads, where a slight disintegration of the surface had begun, further destruction could be retarded and the road maintained through another season in good condition by a single application of road oil with the addition of a thin coat of stone screenings. Relatively better results were secured when two applications were made.

The oil was purchased by the car load. Without heating, it was distributed on the road with an ordinary sprinkler having an oil-distributing attachment. The cost, including the screenings, was about 8 cents per sq. yd. This work, while giving good results, was not entirely satisfactory on account of the uneven distribution of the oil caused by variations in the speed of the horse-drawn sprinkler. These speed variations were due to change of grades and the passing from a maximum to a minimum load to be hauled, and also to the uneven oil pressure maintained at the distributing point on account of the constantly changing head. This objectionable feature was entirely eliminated by the use of an auto-sprinkler owned by the oil company. With this sprinkler perfectly satisfactory results were obtained, the machine being capable of traveling at uniform speed, regardless of changes in grade or load. The varying pressure due to head of oil was overcome by maintaining uniform air pressure in the tank. This pressure added materially to the final result by forcing a portion of the oil into the body of the road. Almost double the quantity of oil can be used, without causing small rivulets to flow to the sides of the road, when the distribution is accomplished in this way.

After this first trial, all surface oiling was done with auto sprinklers at a cost of 1.2 cents per sq. yd. of surface treated; but this did not include the cost of the coat of screenings. The cost of furnishing and

* County Engineer of Mercer County, N. J.

Mr. Eppele.

spreading the necessary quantity of screenings was 3 cents per sq. yd., thus giving a total cost of 4.2 cents as compared with 8 cents per sq. yd. when the entire work was done by the County.

This method of surface treatment, although of a temporary nature only, gave very good results, and, when considered in connection with its cost, deserves to rank among the foremost methods of maintaining an already constructed water-bonded macadam road. Great care should be taken, however, in selecting an oil for surface application. If the oil is too thick, it will remain on the surface and will not penetrate the body of the road. When of this consistency, it will adhere to the wheels of passing vehicles, causing more or less discomfort to the traveling public as well as injury to the road surface. On the other hand, if the oil is too thin or too light, it has too great a spreading and penetrating power for the base it contains. The result secured with such oils is a stained surface more or less devoid of any cementing bitumen, making frequent applications necessary. An ideal oil is one which will penetrate and yet have sufficient base to assist in holding together the mineral aggregate. In the writer's judgment, a product containing from 40 to 45% of petroleum asphalt will generally give the best results.

Of course, it is presumed that the surface of the road to which the oil is to be applied is in fair condition, being free from ruts or other pronounced inequalities. If it is not in fair condition, the surface should be renewed before the application is made. In fact, no method of surface renewal should be used for roads in poor condition unless provision is made for spreading a mineral aggregate to a depth of 3 in. or more.

Another method of surface treatment, which gives every indication of producing good results, is the application of a heavy residuum oil, having a specific gravity of about 12° Baumé and a residue of about 90% when reduced to 100 penetration at 77° Fahr., which, when absorbed by the sand coating spread over the oiled surface to a depth of 1 in. or more, depending on the quantity of oil to be taken up, forms a sand asphalt coating which is practically water-proof, dustless, and noiseless, and might be termed an artificial rock asphalt. Unlike the oil product previously referred to, this asphaltic compound, while having no penetrative qualities whatever, possesses distinct binding qualities. It is obtained by the distillation of an asphalt-base petroleum oil.

The method of applying this oil was as follows: The surface of the road was first carefully swept to remove all loose matter, after which water was applied with a sprinkler in sufficient quantity to saturate the road thoroughly, the object being to remove any loose particles remaining on the surface and to deposit them in the existing crevices. Uniform distribution of the oil cannot be accomplished if there is

much loose material on the surface. The oil was received in tank cars, and, after heating to about 150° Fahr., was pumped into a road sprinkler and applied to the washed surface with a special spreading device, using ½ gal. per sq. yd. of surface. When the asphaltic product came in contact with the wetted surface of the road, it was chilled, thus preventing it from running off at the sides. While in this chilled state, it was absorbed by the sand coating which had been immediately spread. The sand used was of a sharp, gritty nature, entirely free from loam and moisture. The road was opened to traffic after a reasonable amount of rolling, and the spreading of additional sand at points showing a surplus of oil. The condition of the surface then improved daily. The value of this method of repairing roads depends on the fact that an oil of this consistency, when covered with a layer of sand, will be slowly drawn to the surface by solar heat; consequently, work of this class should be done only on clear, warm days. Mr. Eppele.

The experimental work done by this method, was necessarily crude, owing to improper machinery, etc., but it is understood that heater tank wagons are now being used which apply the oil under pressure, and thus give more uniform results by eliminating the probability of having a varying thickness of resurfacing material.

It is essential to coat every part of the road with a layer of oil of uniform thickness, as points not coated will have no covering, and those receiving an excess of compound will become wavy by the rolling of the surface under extreme loads. It is also apparent that the surface of a road to be treated by this method should be free from inequalities, thereby removing all possibility of water collecting in pools on the surface. Although no experiments have been made along this line, it is believed that if a light coating of pea-size broken stone is applied first, and then covered with sand, the result will be a great improvement, giving a better and more lasting covering, which will reduce to a minimum the tendency to creep under heavy loads. The asphaltic oil-sand covering which has been in use for one year indicates that fairly good results will be obtained for another season. It is proper to assume, then, that, under normal conditions, the life of roads resurfaced by this particular method is 2 years, after which, with the addition of a lighter coating, it is claimed that the life can be extended to 4 years. The cost of the compound was 5 cents per gal., delivered, and the total cost for work of this class was about 9 cents per sq. yd., or an average of 4½ cents per sq. yd. per year.

By far the greater part of all resurfacing work on roads in New Jersey during 1910 was done, or at least started, under a State specification, which called for the spreading of 4 in. (loose measurement) of 1½-in. broken stone; the coating of the stone by the application of heated asphaltic binder to the surface; thorough rolling; the addition of ¾-in. stone, or screenings, and sometimes sand, and continued

Mr. Eppele.

rolling until the road was finished. Although it was thought that excellent results would be produced by this method, it failed to come up to expectation.

A large number of contracts for new construction, as well as reconstruction work, had been entered into throughout the State, when, in the middle of the season, it was decided, with the consent of all interested parties, to change the contract by eliminating the use of the asphaltic binder specified and by providing for the construction of a water-bonded macadam, and the application, when completed, of "No. 4 Road Oil" to the surface. In every case this change carried with it an allowance or rebate, on the part of the contractor, to the State and County, and, at the same time, gave greatly improved results, when compared with those previously obtained.

That part of the original specification relating to the qualities of the asphaltic binder to be used was prepared by an expert chemist, and called for heating the compound, which was shipped in tank cars, to 200° Fahr., after which it was to be applied to the road by a sprinkler.

It is claimed, by the parties who furnished the asphaltic binder, that the cause of failure was due, not to the chemical proportions, but to the physical properties of the compound, and that if a hard binder, without being "cut back" or fluxed with a distillate, had been specified to be applied by hand after heating to 350° Fahr., the final results would have been entirely satisfactory. This claim by the oil company, however, is not in accordance with the results obtained in New Jersey during the past 3 years.[8] The binder used during 1909 was not a "cut back" product, yet the results were worse than those secured by the "cut back" compound specified in 1910. Both products were of about the same consistency. According to the opinion of the chemist referred to above, one of the causes of failure was the use of an oil having a paraffine base, oils of this type appearing to lose their binding properties much more quickly than those having an asphaltic base.

It is possible, however, with a proper asphaltic binder, to make a satisfactory bituminous macadam road by the foregoing methods, as shown by some of the roads constructed; but, to secure these results, it is essential to have experienced men, a satisfactory distributor, and a reliable contractor. A hard binder, heated to 350° Fahr., and applied by hand, should not fail to give good results.

Although the results obtained after changing the method of construction, as related above, were pleasing, it should not be understood that, in the opinion of the State Road Department of New Jersey, or the various county engineers, this is the most desirable method for future construction work. It is the writer's opinion that a bituminous binder of some kind should be used, and that a change to the old water-bonded macadam, even if it provided for the application of a light dust-laying oil to the surface, would be a step backward.

Mr. Eppele.

The State Road Department, however, was in a peculiar position, which, in justice to it, should be mentioned. It is understood that, in an opinion given by the Attorney-General of the State to the Road Department at the time a change of method of construction was being considered, it was held that the Department would be justified in modifying the specifications where the sole object was to improve the results after construction, provided such change did not involve an addition to the original cost; but that a change at an extra cost would not be legally justifiable without canceling existing contracts and re-advertising for bids. Under these circumstances, there was no other course to follow than the one adopted.

The average cost of resurfacing work under the original specifications was 58 cents per sq. yd., which cost included the necessary work to be done on shoulders. Deducting the average rebate of 6 cents per sq. yd. leaves a net cost of 52 cents per sq. yd. for work of this class.

Mr. Rablin.

JOHN R. RABLIN, M. AM. SOC. C. E. (by letter).—In the consideration of the subject of macadam and gravel road construction and maintenance, greater importance is generally given to the problems of construction than to those of maintenance. Although the methods used and the results obtained in the construction of the road surfaces have, without doubt, great bearing on their future maintenance, nevertheless, the efforts and expense which are put into the proper construction are of little value from the standpoint of the durability of the surface, if the maintenance work is neglected.

The problems of maintenance have become very much more difficult during the past five years, and especially that of the preservation of the road surfaces constructed without bituminous binders to serve the conditions of traffic of eight or ten years ago. It has been demonstrated, however, that it is not absolutely necessary that the surfacing shall be constructed with bituminous binders in order to withstand the traffic conditions of to-day, as many miles of roadway surfaced with ordinary macadam and gravel, by early and constant attention to maintenance, have been kept in good condition for the last five or six years by using surface treatments with bituminous materials, and without any extensive work of resurfacing.

In surface treatment there is no doubt that some advantage and benefit is obtained if the roadway surface is constructed with a bituminous binder, and such a surface can probably be maintained more economically, as more wear can be allowed without danger of disintegration, making the re-treatments less frequent and requiring less material. The results obtained on work of surface treatment with tars and oils which has been under the direction of the writer, have been generally very satisfactory, on ordinary macadam and gravel surfaces, and on those constructed with bituminous binders.

Mr. Rablin.

The work referred to was begun in August, 1906, when about 67 500 sq. yd. of macadam roadway surface were covered with Tarvia. Previous to this work, very little expense had been necessary on these roads, except the cost of watering, as they had been built only from four to five years. The cost of watering alone averaged about 3½ cents per sq. yd. of surface per year. The cost of this first work of surface treatment with Tarvia averaged 6⅔ cents per sq. yd., including labor and materials. The details of this cost are as follows:

Tarvia	$0.0262 per sq. yd.
Stone screenings	0.0184 " " "
Labor	0.0220 " " "

The labor item includes preparing the roadway surface and applying the tar and screenings.

When it is taken into consideration that this work acted as a substitute for re-surfacing, which would otherwise have been necessary, and also eliminated the sprinkling with water, it is evident that it was an economical method of road preservation, provided it would be effective for at least one year. That this has proven to be a fact will be shown by the costs of maintenance of this and other sections from the time of the first treatment to the present. For the following year, 1907, the total cost of repairs and re-treatments of the Tarvia section averaged 3½ cents per sq. yd. for the 67 500 sq. yd. During this year about 150 000 sq. yd. additional macadam road surface were treated in the same manner at a cost of from 5¾ to 9 cents per sq. yd., or at an average cost of 6½ cents, including labor and materials.

Besides this additional Tarvia work, experiments were made with other materials during 1907. About 225 000 sq. yd. of roadway—one-quarter being gravel-surfaced and three-quarters macadam-surfaced—were covered with a mixture of crude water-gas tar and asphaltic oil. These materials were mixed in different proportions of 2, 3, and 4 bbl. of oil to 6 bbl. of tar. The results obtained with this material were more in the nature of dust suppression than surface preservation, as it contained but little actual binding properties. Better results were obtained with it on the gravel surfaces than on the macadam. The average cost was 4 cents per sq. yd., including materials and labor for one application.

Asphaltoilene, an asphaltic oil, was used on about 35 000 sq. yd. of roadway, one-half macadam- and one-half gravel-surfaced. This material was furnished and applied for 6 cents per sq. yd., the only other expense being the sand covering, which was very small. The results obtained by the use of this material were very satisfactory and economical, as in the case of the gravel-surfaced road it was effective for two years, with practically no expense for repairs during

that period, and the macadam roadway was maintained during the second year by patching and some slight re-treatments, at an average cost of 3 cents per sq. yd. Mr. Rablin.

During the next year, 1908, about one-half of the total area previously surfaced with refined tar, or 120 000 sq. yd., was retreated with the same material, at an average cost of 4¾ cents per sq. yd., and the remainder kept in good condition without repairs. About 60 000 sq. yd. additional, of this same class of work, were done, except that a different brand of refined tar was used. The average cost of this additional work was 5 cents per sq. yd. About one-quarter of the area surface-treated during the previous year with asphaltic oil was given a light re-treatment at a cost of about 3 cents per sq. yd., and 75 000 sq. yd. additional were laid, at a cost of 7 cents per sq. yd. Of the area previously treated with water-gas tar and asphaltic oil, 53 000 sq. yd. were re-treated with the same material, and 46 000 sq. yd. additional were laid. The cost of both averaged 3½ cents per sq. yd.

A new combination of materials, which has been used to a considerable extent since, was tried in this year on about 30 000 sq. yd. of macadam surface. It consisted of 90% refined tar and 10% residual asphalt. The cost of this work averaged 5½ cents per sq. yd. The results obtained with this material appeared to be superior in some respects to those with the tar alone.

During 1909 and 1910, the tar surfaces have been maintained in good condition generally by re-treatments with the tar and asphalt mixture, at the rate of about once in two years, at an average cost of 6 cents per sq. yd. for each re-treatment, and the asphaltic oil surfaces have required about the same attention and at the same average cost. Where traffic conditions are extremely severe, more frequent repairs are required, and the average cost will vary up to 6 cents per sq. yd. per year. Where the macadam surfacing is constructed with a bituminous binder, the surface, or painting coat, is also required from time to time, but, as stated previously, not as frequently.

It appears from these tests of the past five years that surface treatment of macadam and gravel road surfaces with refined tars and asphaltic oils has been, up to the present time, an effective and economical method for the preservation of road surfaces subjected principally to an automobile traffic, which was the class of traffic under which these tests have been made.

R. K. Compton, Esq.*—The following is a description of the method used in Baltimore of applying a surface treatment of oil, and its cost. Mr. Compton.

Where the original road-bed was in good condition for any material distance, and required no new stone for resurfacing, the surface treatment was tried. The old road-bed was thoroughly cleaned with wire

*Assistant City Engineer, Baltimore, Md.

Mr. Compton. brooms, and all dust, dirt, and foreign material were removed. A coating of Fairfield Anti-Dust, a local material, heated to a temperature of 190° Fahr., was then applied, to the extent of ½ gal. per sq. yd. of surface, and was thoroughly broomed in. About two hours was allowed for the bitumen to set and penetrate, after which the surface was covered with a layer of granolithic from ⅝ to ¼ in. in size, and then thoroughly rolled with a 12-ton roller.

The cost of flushing 3 024 sq. yd. of surface was 13.29 cents per sq. yd., itemized as follows:

Labor	5.00 cents.
Stone	3.00 "
Rolling	1.50 "
Binder	3.15 "
Coal	0.35 "
Hauling binder	0.15 "
Incidentals, including supplies and repairs to plant	0.14 "
Total	13.29 cents.

The surface treatment was only used where the original section was in fairly good condition, in order to prevent raveling, and at the same time to allay the dust nuisance. A recent inspection of the road, after it had been subjected to a heavy fall of snow, a sudden thaw, and freeze, in succession, showed that the surface treatment has served the purposes for which it was intended. Where the machines had worn a rut through the snow and ice to the surface, the road showed some signs of raveling, but this defect can be corrected at small expense in the spring. Every machine and vehicle followed this same rut for weeks, so that there must have been an enormous wear on the surface for a width of only about 6 in.

Mr. Connell. William H. Connell, Assoc. M. Am. Soc. C. E.—A surface treatment of oil and sand was used in connection with experiments on White Plains Avenue, Borough of the Bronx. The description and detailed cost follow:

White Plains Avenue Experimental Pavements.

Section Eighteen: 39 + 22 to 41 + 20.

Standard Oil Company's Sand-Surface Method.

Application of Oil and Sand.—This part of the road was an old strip of water-bound macadam. It was swept, and then Standard Oil Company's Binder A was applied, the quantity being ¾ gal. per sq. yd.

Over this was spread a thickness of about 1 in. of sand, which was lightly rolled. Mr. Connell.

Cost per Square Yard.

Sand course	$0.178
Hauling binder	0.002
Tar heater	0.001
Fuel	0.021
Total	$0.202

Sand Course.

		Per square yard.
Labor	$40.45	$0.086
Sand, 10 cu. yd., at $1.21	12.10	0.026
Binder, 384 gal., at $0.08	30.72	0.066 = 0.82 gal. per sq. yd.
467 sq. yd., at $0.178	$83.27	$0.178

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	14	$4.00	$7.00
Cleaning macadam	3	24	2.25	6.75
Pouring and carrying binder	4	40	2.25	11.25
Heating binder	2	20	2.25	5.62
Spreading sand	3	21	2.25	5.90
Tamping	1	2	2.25	0.56
Rollerman	1	6	4.50	3.37
467 sq. yd., at $0.086				$40.45

The actual time of laying the sand course was 2½ days (gang time), or an average of 187 sq. yd. per day, including cleaning and brooming old macadam.

Temperature, Weather, Etc.

Pavement.	Date, 1910.	Temperature.			Weather.
		9 A. M.	12 NOON.	3 P. M.	
Standard Oil-Sand...	October 18th. Begun.	59°	65°	66°	Clear.
	October 19th	59°	65°	68°	Clear.
	October 21st. Finished	55°	57°	59°	Cloudy.

Arthur W. Dean, M. Am. Soc. C. E.—Omitting from consideration the bituminous material used in the construction of city pavements, it is a fact that a very large percentage of that used on highways is for the surface treatment of macadam roads. The surface treatment prevents old roads from going to rapid destruction; in Mr. Dean.

Mr. Dean. the finishing of new roads under construction, it is used as an expedient to save the road until it can be determined what form of construction will be economical, without the use of a surface treatment every year, or every two or three years, as the case may be.

In Massachusetts during 1910 about 225 miles of road were treated with bituminous binders, of which approximately 200 miles were treated with materials which required heating before application. This form of treatment, that is, the use of heavy oils or tar, is considered much more effective and economical, in the course of two or three years, than the lighter oils or binders. The average cost of surface treating roads in Massachusetts during 1910 was approximately $7\frac{2}{3}$ cents per sq. yd., the width of the treatment being ordinarily 15 ft.

The details of the cost of this surface treatment may be of interest. Nearly all the oil and tar was purchased in tank cars fitted with steam coils, so that the material could be easily and conveniently heated. The expense of heating varied from about $\frac{1}{4}$ to about $\frac{1}{2}$ cent per gal., the larger cost being the result of the necessity, at times, of keeping up the heat in a car for several days on account of unavoidable delays. The cost of preparing the road surface varied from $\frac{1}{2}$ cent to 2 cents per sq. yd. The cost of heating and applying the oil, where the average haul was 2 miles, was about 1 cent per sq. yd. The material cost from 1 cent to 3 cents per sq. yd., according to the quantity of oil or tar used. Sand covering cost from 2 to 4 cents per sq. yd.

It is usually impossible to close the highways to travel while the material is being applied, hence it is customary to clean the entire road, then apply the oil on one side only, and immediately cover it with fine gravel, coarse sand, stone chips, or pea stone. Traffic is then allowed on this completed side while the other side is being done. The sweeping is done with an ordinary horse sweeper fitted with a rattan broom. Any patching is done by painting the depressions with the bituminous material and immediately applying a small quantity of $\frac{3}{4}$-in. stone. The temperature to which the material is heated before applying varies from 180° to 250° Fahr., depending on its density.

Some roads oiled in 1908 with heavy asphaltic oil are almost as good to-day as they were then. A small amount of patching has been necessary, which has cost $\frac{1}{2}$ cent per sq. yd. for the entire period, this cost being based on the entire area of the road. On the other hand, there is one road which was treated with a $\frac{1}{2}$-gal. coat of heavy asphaltic oil early in August, 1910, and early in October all the bituminous material broke up and had to be removed. In this instance the road was traveled by many heavy ice wagons, and their narrow tires flaked the material so that it was of no use whatever, and in wet weather was a nuisance.

Mr. Dean. It has been found that, on the surface of roads, much better results are obtained by applying bituminous binders under pressure, and, if it is intended to use ultimately ½ gal. per sq. yd., to make two applications of ¼ gal. per sq. yd., covering each application with a sufficient quantity of gravel or sand. The traffic is allowed on the road for about a week before making the second application. This seems to give a more uniform mixture of the oil and sand on the surface, and the application of the material under pressure seems to make it less likely to peel up, as it often does where it is applied without pressure, and with a coating of ½ gal. at one time.

Mr. Ellis. Fred. E. Ellis,* Esq.—The following method was used a few years ago in a large New England city: A bottom course of 2½-in. broken stone, 4 in. in thickness after rolling, was laid and treated with about 1½ gal. of bituminous material per sq. yd. On this surface 1½-in. stone was laid to a thickness of 2 in. after rolling, and treated with ¾ gal. of bituminous material per sq. yd. A sealing coat of bituminous material was then applied and covered with pea stone, which was rolled in. This method was used throughout the year, about 15 000 tons of broken stone being used on the roads in that time.

It was found that roads treated in this way were expensive to keep in repair. In warm weather it was necessary to put on sand frequently to absorb the bituminous material. During the fall and winter, the top course picked up, and after this course had become loose it was impossible to stop the raveling without an application of more bituminous material. During the present year the top course of stone was omitted, only one course of 2½-in. stone being used. The road was constructed as for ordinary water-bound macadam, with one layer of stone bound with stone dust which passed through a $\frac{3}{16}$-in. screen. These roads were opened to traffic as soon as they were flushed. When sufficient macadam had been laid to use the contents of one tank car, the surface was swept clean and about ½ gal. of bituminous material per sq. yd. was applied; the road was then covered with screenings and rolled thoroughly. The screenings consisted of the pea stone and the dust mixed. This method of construction has done away with all raveling, and is thought to be the proper way to build roads in that city.

Mr. Parker. Harold Parker, M. Am. Soc. C. E.—Several years ago, in England, the speaker saw a machine for distributing bituminous material on roads under pressure, and was so well pleased with its work that two of these machines were purchased and brought to Massachusetts, where they have been used steadily since that time. Some similar machines have been invented by his department which do the work as well, if not better.

* Manager, Essex Trap Rock and Construction Co.

Mr. Parker. The speaker felt that oil or tar should be applied to the surface of a macadam road in a very thin layer and under pressure. The result of a great number of experiments showed that the road should be moist when the oil is applied. The general method of procedure is as follows: The road is made perfectly clean by washing the surface with a hose or sprinkling cart. All inequalities are patched, thus restoring the original cross-section. While the road is still moist, about ¼ gal. of oil per sq. yd. is distributed over the surface very evenly by the spraying machine under a pressure of about 70 lb. The road is at once completely covered with sand or stone chips, which are rolled with a 10-ton roller, after which the road is ready for use.

Although there is probably no penetration of the oil into the surface of the road (below the old surface), the stone or sand which is put on makes a very complete mixture with the oil, and the oil is distributed uniformly through it. The next layer is placed in a similar manner, so that a very much better result is obtained than if all the oil were put on at one time with enough gravel or stone on top to absorb it. The success of this method of treatment depends on only two things: that the proper kind of oil is used, and that it is applied on a clean road. In Massachusetts the oil has to comply with specifications which are based on the results of more than 600 chemical and physical tests of oils which have been used under different conditions.

Mr. Driscoll. Michael Driscoll, Esq.*—In small towns, it often happens that while it is desirable to use bituminous materials, it is not expedient to go to the expense of purchasing a spreading machine as elaborate as the English type, and it is not always convenient for the producers to send out any equipment which they may have. The American Tar Company's distributor has been used in the Town of Brookline, Mass., and gives a very even distribution of the material. The machine is of the gravity flow type and consists mainly of a triangular trough mounted on wheels, the trough being connected with a series of pipes by which the distribution is effected. It is possible to maintain a uniform head of material in the trough, and thus avoid the unequal distribution which comes from using watering carts, because of the varying head.

Some Gulf A oil was applied on one of the streets in Brookline in 1910, using the distributor just described. This street is, perhaps, as important as any leading out of Boston; it has two roadways, one 50 ft. wide, and the other 30 ft. wide, with a reservation between for street cars, and is subjected to a large amount of mixed traffic. On that part of the road most used by automobiles, the material ironed out perfectly under the traffic, but where the horse-drawn traffic predominated, the surface rutted in wet weather and was generally in poor condition.

*Superintendent of Streets and Sewers, Brookline, Mass.

Mr. Driscoll.

A similar condition was noticed on one of the double-road boulevards of the Metropolitan Park roads, one side of which was devoted to horse-drawn traffic and the other to automobiles. In going over the road limited to automobile traffic one day, the speaker was surprised to find that a part of it was disintegrated in many places and on the point of going to pieces. The material was very similar to that used on Beacon Street; two or three blocks further on, he found that the other road was being repaired, and that the horse-drawn traffic had been diverted to the road devoted to automobiles, with the result above described.

Mr. Sharples.

PHILIP P. SHARPLES,* ESQ.—In 1910 two types of English tar-spraying machines were purchased and brought to the United States. One, obtained by the Massachusetts Highway Commission, is towed by a steam roller or traction engine, and spreads the tar on the road through nozzles by direct pump pressure. The other, purchased by the Barrett Manufacturing Company, is an automobile, and spreads the tar through nozzles by pumping it into an intermediate cylinder against air pressure.

Each machine worked very well indeed on smooth roads, and especially on those which had been previously tarred, but, on roads which were being treated for the first time, the machines were not as successful. Where the road has once received a coating of bituminous material, the next coat finds a surface to which it can adhere very easily. For this reason it is preferable to apply bituminous material in several thin coats than in one heavy coat of the same total quantity. These heavy-pressure tarring machines, although they work very well in England, and have stood the test of working conditions for four or five years, have not proved adaptable to American road conditions. The roads in the vicinity of Boston have the reputation of being as good as anywhere in the United States, but the machine which was purchased by the Barrett Manufacturing Company, after being used for a year on these roads, racked to pieces and was consigned to the junk heap.

Many other types of machines have been tried in construction work, in an endeavor to get one which would spread tar materials evenly over broken stone. The problem is much more difficult than in the surface treatment of roads, because the starting and stopping of machines on a broken stone surface (not always thoroughly consolidated), makes it very difficult to get an even distribution. The lapping of the strips laid down by each passage of the machine causes further trouble.

Owing to these difficulties, the speaker's experience indicates that a machine which uses simply a hand hose is preferable to any other type for construction purposes, and, up to a certain point, is also as

*Chief Chemist, Barrett Manufacturing Company, Boston, Mass.

Mr. Sharples. well adapted for surface spreading. It is necessary, however, to have a nozzle which will atomize the bituminous material thoroughly, in order that the stone may be properly coated, and it is necessary to spray the material under pressure in order to atomize it thoroughly and distribute it properly. In constructing macadam roads, it is almost impossible to lay the stone so that a uniform surface will be obtained throughout, but a competent laborer, working with a hand spray hose, can make allowances for the non-uniformity of the surface, and obtain an even distribution of the bituminous material.

One very important point in making surface applications of bituminous materials is a thorough study of the traffic conditions on the road to be treated. Too little attention has been given to this point, and therefore the same materials have not produced the same results in different experiments. A difference in the numerical ratio between horse-drawn and automobile vehicles calls for a radical change in the surface treatment in order to be successful. Provision can be made for heavier traffic conditions, up to a certain point, by increasing the viscosity and weight of the bituminous material which is to be applied. When the surface layer becomes too thick, however, it breaks down under the weight of a heavy traffic. When this point is reached, the bituminous material should be made so thin that it will penetrate into the roadway, leaving the stone on the surface as a mosaic to take the wear of the traffic.

A point is soon reached, however, where a surface treatment, no matter how efficacious, cannot stand the strain put upon it. The remedy then is a more thorough construction by the use of a bituminous binder throughout the body of the road. The surface treatments will still be found necessary, in order to keep the surface in good condition at a minimum annual expense.

Mr. Bishop. Hubert K. Bishop, M. Am. Soc. C. E.—In New York State, three roads near Rochester have been re-surfaced, under the speaker's direction, with Kentucky rock asphalt, and in Erie County two were surfaced with that material on the original construction. All but one of these roads have proved very successful, and the speaker believes that the failure of portions of one of them was due to faulty construction of the foundation and the drainage rather than to the material itself. On the roads thus re-surfaced the cost of maintenance has been exceedingly low. The roads are very smooth, have the appearance of asphalt pavements, and are not slippery to any great extent. One of the roads in Erie County was about 7 miles long, and was repaired in a few bad spots shortly after it was constructed, but since that time it has been in excellent condition. The analysis of the asphalt shows from 6 to 9% of bitumen.

One piece of Kentucky rock asphalt, about 500 ft. long, was put on the main street of Buffalo, about 4 years ago, on an old macadam

foundation. It has always been in very good condition, and has not cost much for maintenance. The cost of construction was rather high, being about 50 cents per sq. yd. more than that of an ordinary macadam road. In the use of rock asphalt there is no material increase in the labor item over that of ordinary macadam. Mr. Bishop.

In re-surfacing roads, the whole foundation was cleaned and sometimes scarified on the edges, and then a thickness of about 3 in. of stone of uniform size was laid. The natural rock asphalt was delivered in carts in the form of screenings. This material sometimes had to be picked loose, loaded into wagons, and drawn to the road, where it was run through special rollers for pulverizing and making it pliable. On the course of stone first laid it was then spread to such a depth that, after rolling, the thickness would be 1 in. The roller was usually kept working on the road for some time after the material was applied, and until it was thoroughly compacted and hard.

R. A. Meeker, Esq.*—In New Jersey, when a road is worn to such an extent that the cross-section has been destroyed and the crown has been lost, the road is always scarified and brought up to the proper crown, and then thoroughly rolled and consolidated before any new surface material is placed on it. The top dressing is next added, together with the lighter or heavier asphalt-oils, and in some cases tar. The best results have been obtained with a light application of oil and sand, or oil and screenings, using not more than 1 in. of material at a time. Oils from almost every section of the country have been used in the work, those having an asphaltic base giving better results than those having a paraffin base. The success of the road, however, depends to a large extent on the care with which the work is done. Mr. Meeker.

Another method was to build a good, firm, thoroughly-consolidated, water-bound macadam, and then apply a 40 to 45% asphaltic oil at the rate of $\frac{1}{2}$ gal. per sq. yd. This gave very good results only when the stone was thoroughly consolidated.

In the southern portions of the State the gravel roads are maintained by the same methods, except that, instead of scarifying them, they are harrowed. The gravel is then put on, and in some cases light oil (from $\frac{1}{4}$ to $\frac{1}{2}$ gal. per sq. yd.) is put on the road after it has been thoroughly consolidated. If applied on loose material the oil acts simply as a lubricant, and not as a binder.

James Owen, M. Am. Soc. C. E.—Experience during the last two years (particularly 1910) has tended to crystallize the practice of road construction and maintenance into more tangible shape. Experimental experience, unfortunately, has been too limited to afford the requisite data for the satisfactory solution of the problem, because the results Mr. Owen.

* State Road Supervisor of New Jersey.

Mr. Owen.

of general practice based thereon have not been uniform, and in some cases there have been disastrous consequences.

In all road construction the actual manipulation of materials and labor are paramount, and when a substitution of skilled for ordinary labor is made, the enhancement of the cost is great, and becomes a vital question for the community at large, in the matter of taxes.

In the accepted practice in the part of New Jersey in which the speaker is more particularly interested, the original cost of construction for an 8-in. Telford road varied from 50 to 80 cents per sq. yd., and the maintenance cost, prior to the automobile era, was 3 cents per sq. yd. per annum. The increase of use, due to motor cars, and the higher price of labor, have raised this cost to 5 or 6 cents per sq. yd. The original estimate for the extra cost of bituminous material, according to Mr. Blanchard, was about 12 cents per sq. yd., with a presupposition of success. The speaker found that bids for penetration work were at about that price, but, owing to the non-success of one contract on those terms, he has had to abandon all contracts for public roads, based on penetration methods, and those made prior to that time have been changed to surface oiling.

In the maintenance department, with ordinary unskilled labor, but with good, old-fashioned road manipulation, the penetration method was absolutely disastrous. The work itself was costly, and, except in spots, there was not even a semblance of a smooth surface; therefore that system was abandoned.

In the speaker's district, an examination of roads surfaced by the penetration method shows that every mile thus treated requires re-surfacing and renewals. In many places there is creeping and crawling of the material, in other places there are deep ruts, and most of the remainder is broken.

The speaker's experience with the mixing method is limited, and though a very satisfactory surface can be obtained by this process, the question of cost must be considered, as almost all the processes are patented, and the prices, for renewals of surface 2 or 3 in. thick, vary from 80 cents to $1.25 per sq. yd. The economy of this process can only be decided after a lapse of years. Comparing 6 cents per sq. yd. with $1.25, a period of 20 years is necessary to make things even.

The speaker's practice at present is to build all roads of Telford by the old water-bound process, with moderate packing; then, as soon as this surface is smooth, a coating of 40% asphaltic oil is spread, and two coats of oil are applied during the maintenance year. In making general repairs in rural sections, he applies two or three coats of oil to ordinary water-bound macadam. In villages and districts of concentrated travel roads are re-surfaced by the mixing method. In cities and large towns, close-grouted granite blocks are substituted for the old macadam. For dust suppression the speaker uses from 35 to 40%

of asphaltum, as any greater percentage causes the surface to be pitted, and in some cases to peel badly. Mr. Owen.

Charles W. Ross, Assoc. Am. Soc. C. E. (by letter).—There are on the market a great many kinds of tars and asphaltic oils, any one of which might be used with advantage for road surfaces. The writer cannot agree with all that has been said in regard to the methods of applying such materials. With reference to the different machines used for this purpose, there may be some choice, but the work can be done satisfactorily with any one of them. Several methods have been tried in Newton, Mass., but the writer believes thoroughly in heating the sand and oil and applying the mixtures in thin layers, as the results have been very satisfactory; but, whatever the treatment to be used or the method of applying the materials, it is important to bear in mind the necessity of keeping the road surface swept clean. This may be done by hand or by a horse-sweeper, the latter being the cheaper. When the surface is clean, the work can be done at an expense which is slight, considering the benefits derived, especially on city streets. Mr. Ross.

The writer believes there will be an increased use of the heavier asphaltic oils rather than of the lighter emulsions. In Newton, in most cases, the various tar preparations have given satisfactory results, especially on macadam, where they have been much better than on gravel surfaces. The action of frost has more or less to do with tearing up such surfaces, and, when once broken, it is almost impossible to bind them together again.

The methods of applying these different preparations have been discussed frequently, but in any case the road should be cleaned thoroughly first, and, after the oil has been applied, every effort should be made to keep the street surface as clean as possible. In Newton, this is done by what is called the "block system." Each man is required to keep about 2 miles of street clean during the entire season. He not only sweeps the surface, but patches and fills all inequalities, cuts the grass along the borders, keeps the edges trimmed and the sidewalks neat; in fact, he does all the repair work on his section, exclusive of the general spring cleaning given all streets, regardless of whether they have oil, tar, or macadam surfaces.

Probably many of the lighter oils, such as dustoline and tasscoil (a new preparation which has lately come on the market), will be used extensively on the side streets of Newton, where the traffic is not heavy, and, in the writer's opinion, will prove satisfactory in keeping down the dust. These materials are merely for dust-laying purposes, but their cheapness will doubtless appeal to officials in many cities.

(6) THE USE OF BITUMINOUS MATERIALS BY PENETRATION METHODS.

By Messrs. W. W. Crosby, Charles W. Ross, R. K. Compton, William H. Connell, Michael Driscoll, Malcolm H. Smith, E. H. Rogers, Alexander F. Armstrong, H. G. Shirley, Paul D. Sargent, R. A. Meeker, and James C. Wonders.

Mr. Crosby.

W. W. Crosby, M. Am. Soc. C. E.—Up to the present time, local conditions have seemed to require the use of the penetration method in the treatment of the State roads of Maryland with bituminous materials. The speaker will report only on the work directly under the supervision of the State authorities.

In 1909 Park Heights Avenue was treated with 15 different bituminous materials in 29 sections. The details of the treatments were reported to the Second International Road Congress, and this report has been reprinted in some American publications, so that repetition will be avoided here. Heretofore, circumstances have prevented giving the costs of the 1909 work, therefore tables of cost data are submitted herewith.

The method generally used was to put down the macadam in the ordinary way up to the point of beginning the binding, as usually practiced, with water and stone chips or screenings. Slight variations from this method have been made. For example: In a few instances the courses of the macadam have been reversed and the "Number ones" placed on top, and the "Number twos," as ordinarily called, placed in the bottom. In some cases, before the pitch was applied to the stone (which had been thoroughly rolled), a layer of sand, to fill partly the voids between the stones, was put on and rolled in. In other cases the pitch was applied to the clean rolled second course.

The quantity of pitch used has varied from $\frac{3}{4}$ gal. to as much (in one particular instance) as 5 gal. per sq. yd. The use of 5 gal. was an accident, which has brought about a very curious result which is now being watched. It is too early to report at the present time, however, on the success or non-success of this particular road.

The maintenance cost data shown in Tables 1 and 2 include taking care of the bleeding by putting on chips, taking care of the slipperiness, which was objectionable on some sections, and in one case it covers the removal of a very disagreeable mud which developed on the surface in the early spring following its construction.

Mr. Crosby.

TABLE 1.—Construction and Maintenance Cost Data on Eighteen Sections of Maryland Roads.

Section No.	Area, in square yards.	Material used.	Time when used.	Cost per square yard of resurfacing macadam.	Cost of pitching, including chipping.	Cost per square yard of maintenance, to 1/1/11.	Total cost per square yard of treatment, to 1/1/11.	Total cost per square yard of road surface, to 1/1/11.	Remarks.
1	11 226	Texaco.....	7 and 8/09	$0.337	$0.327	$0.088	$0.415	$0.752	
2	1 936	Gulf.....	8 and 9/09	0.339	0.434	0.080	0.514	0.853	
3	1 243	Texas.....	9/09	0.336	0.418	0.187	0.605	0.941	
4	1 173	Fairfield...	9/09	0.337	0.449	0.081	0.530	0.867	
5	1 712	U. G. I.....	9/09	0.339	0.344	0.081	0.425	0.764	
6	1 908	Warren....	9 and 10/09	0.333	0.606	0.079	0.685	1.018	
7	1 909	Tarvia.....	10/09	0.337	0.618	0.082	0.700	1.037	
8	1 816	Tarite.....	10/09	0.336	0.605	0.080	0.685	1.021	
9	1 264	U. G. I.....	11/09	0.340	0.454	0.091	0.545	0.885	
19	128	S. O.....	11/09	0.241	0.174	0.087	0.261	0.502	Mat. gratis.
20	1 294	U. G. I.....	10/09	0.230	0.408	0.088	0.496	0 726	
21	694	Texas.....	10/09	0.228	0.551	0.088	0.639	0.867	
22	977	Gulf.....	10/09	0.229	0.405	0.082	0.487	0.716	
23	845	Warren....	10/09	0.224	0.691	0.082	0.773	0.997	
24	2 246	U. G. I.....	9/09	0.229	0.257	0.081	0.338	0.567	
25	1 532	Fairfield...	9/09	0.228	0.353	0.057	0.410	0.638	
26 and 27	5 207	Local and U. G. I.	8/09	0.216	0.303	0.057	0.360	0.576	

TABLE 2.—Construction Cost Data on Eight Sections of Maryland Roads.

Section No.	Area, in square yards.	Material used.	Time when used.	Cost per square yard for resurfacing macadam.	Cost of pitching, including chipping, per square yard.	Total cost of road surfacing, per square yard.
10	3 532	U. G. I.....	May and June, 1910.....	$0.397	$0.242	$0.639
11	5 981	Texas.....	June, 1910.....	0.397	0.264	0.661
11-A	1 261	Mixed Tar.....	June, 1910.....	0.397	0.262	0.659
12	4 685	Headley.....	June and July, 1910..... ...	0.397	0.327	0.724
13	4 203	B. A. P.....	July and August, 1910.....	0.397	0.325	0.722
14	3 184	Fairfield.....	August and September, 1910	0.397	0.292	0.689
15	5 808	Fairfield Anti-Dust.	October, 1910.....	0.397	0.084	0.481
16	1 100	U. G. I. Anti-Dust..	September and October, 1910	0.397	0.140	0.537

Mr. Crosby.

The traffic on this road has been counted as follows:

Section No. 1, City End.

One-horse vehicles	245	Average per day of 12 hours.
Two-horse vehicles	104	
Three-horse vehicles	10	
Four-horse vehicles	14	
Five-horse vehicles	..	
Six- (or more) horse vehicles	1	
Motor cycles	8	
Motor runabouts	26	
Motor touring cars (4 or 5 seats)	141	
Motor touring cars (6 or 7 seats)	66	
Motor wagons	4	

At times, these figures are greatly exceeded, such as in periods when there is racing at the course, the entrance of which is at the end of Section 3.

Sections Nos. 1 to 3, inclusive.

One-horse vehicles	195	Average per day of 12 hours.
Two-horse vehicles	101	
Three-horse vehicles	3	
Four-horse vehicles	19	
Six- (or more) horse vehicles	1	
Motor cycles	5	
Motor runabouts	15	
Motor touring cars (4 or 5 seats)	141	
Motor touring cars (6 or 7 seats)	55	
Motor wagons	7	

Sections Nos. 14 and 15.

One-horse vehicles	52	Average per day of 12 hours.
Two-horse vehicles	23	
Three-horse vehicles	3	
Four-horse vehicles	4	
Six-horse vehicles	1	
Motor cycles	7	
Motor runabouts	11	
Motor touring cars (4 or 5 seats)	94	
Motor touring cars (6 or 7 seats)	39	
Motor wagons	3	

Sections Nos. 16 and 17. Mr. Crosby.

One-horse vehicles	31	Average per day of 12 hours.
Two-horse vehicles	19	
Three-horse vehicles	3	
Four-horse vehicles	1	
Six-horse vehicles	1	
Motor cycles	3	
Motor runabouts	6	
Motor touring cars (4 or 5 seats)	69	
Motor touring cars (6 or 7 seats)	40	
Motor wagons	4	

Some other penetration work has been done, and some, such as on road surfaces of gravel and of oyster shells, is especially interesting. The speaker regrets his inability to report on this work at the present time.

CHARLES W. ROSS, ASSOC. AM. SOC. C. E. (by letter).—In addition to a large amount of work by surface treatment, the penetration method has been used in Newton to some extent. Mr. Ross.

Several roadways were treated with preparations of 90% Heavy Standard Oil and with Tarite-Asphalt by the penetration method. The materials were heated sufficiently to make them flow freely through the sprinkler of the oil distributor. The macadam surface of the roadway was first spiked up and then covered with a 2-in. layer of screened stone. This stone came from an ordinary gravel pit, and varied in size from ½ to 2 in., the smaller stones filling in between the larger ones. The surface was rolled smooth and hard, after which the oil was applied hot. Used at the rate of about 1 gal. per sq. yd., the oil readily penetrated the entire mass. A coating of gravel which had passed through a ½-in. screen was then applied and thoroughly rolled with a steam roller. The road was opened to traffic within 12 hours after completion. The cost of work of this class varied from 21 to 23 cents per sq. yd., including all labor and materials.

Early in the spring of 1908 one of the principal streets was so badly out of shape that it required a new surface of about 3 in. of stone. It was decided to bind the new material with Tarvia A by the penetration process, and an area of 6 290 sq. yd. was surfaced at a cost of 46 cents per sq. yd. At about this time another street, having an area of 9 821 sq. yd., was treated in like manner with Tarvia A, except that a much thinner coating of stone was applied. The cost of this latter construction was 28½ cents per sq. yd.

Mr. Compton.

R. K. Compton, Esq.*—The following discussion will describe the methods of constructing bituminous macadam roads by the penetration method in Baltimore, Md. This experience extends over a period of 2½ years, but the following description pertains particularly to a stretch of road 1.55 miles in length, from 18 to 32 ft. in width, and having a total area of 17 788 sq. yd. The original roadbed was of ordinary macadam 6 in. thick. It had been in use about 4 years, and was badly worn in places. The entire stretch of road is subjected to heavy automobile traffic, and the portion west of the Stoney Run Bridge, particularly the north drive, is subjected to heavy team traffic from an adjacent sand and gravel plant. The team traffic, though slow moving, is poorly distributed, owing to the usual bad judgment exercised by drivers.

Where the original roadbed was in such condition as to require re-surfacing with new stone, the following method was used: The surface was first swept clean with the ordinary street-sweeping machine, then "spiked" with the roller, picked and raked by hand, and all dirt and foreign material removed. It was then rolled to a firm bed, and sufficient new stone was added to bring the surface to the proper cross-section. In no case was the thickness of new stone less than 3 in. after rolling, nor more than 6 in., so that the average thickness was about 4½ in. Where sufficient depth was available, a layer of stone in sizes from 2 to 1 in., known as No. 2, was first spread and then covered with stone dust in sufficient quantities to fill all interstices. Great care was used in this operation, as it was realized that if the bitumen penetrated the No. 2 course a slack roadbed would result. In order to secure the filling of all interstices, the dust was swept in, and water was used where necessary. Where water was used, however, this course was allowed to dry before placing the next, or No. 3 course.

After the No. 2 layer was completed a 3-in. layer of No. 3 stone, ranging in size from 1 to ¼ in. was laid. Where only the minimum depth of 3 in. was available, this size was used alone. The method of rolling was the same for all courses as with ordinary macadam, except that the last or No. 3 course was rolled firmly but not tightly, the idea being to allow the bitumen to penetrate not less than 2½ in.

After the macadam had been rolled, the binder was applied at the rate of not less than 1½ nor more than 2 gal. per sq. yd. of surface. Two materials were used; namely, "Fairfield Anti-Dust," a local material, which was applied at a temperature of not less than 190° Fahr., and "Standard Asphalt Binder" "B," which was applied at a temperature between 300° and 400° Fahr. An ordinary asphalt

* Assistant City Engineer, Baltimore, Md.

thermometer was used, and the binder was at all times applied at the temperature recommended by the manufacturer. Mr. Compton.

In heating the binder two kettles were used, one having a capacity of 200 gal., and the other a capacity of 400 gal. Both kettles were mounted on wheels, and were easily moved. The smaller kettle worked more satisfactorily with the "Fairfield Anti-Dust," which material runs freely from the barrel and is easily heated, while the larger kettle worked more satisfactorily with the "Standard," which does not run freely from the barrel and requires more heat, thereby requiring more storage capacity, if the work is to be handled economically. A 1½-in. hose, with a spreader on the extreme end, was attached to the rear of the kettle, the length of the hose being from 10 to 15 ft., depending on the width of the road. The binder was applied with this hose, and the flow was regulated by a valve on the rear of the kettle. A rubber hose was first tried, but it was found that it burned out too frequently under the extreme heat of the bitumen. It was then found that the ordinary linen hose, worth about 15 cents per ft., answered the purpose, and was, of course, much more economical. A more uniform distribution of the bitumen was obtained by using the hose than by hand-pouring. The former method is also more economical than hand-pouring, as the labor of heating and spreading bitumen, spreading Granolithic, and rolling it was 4 cents per sq. yd., as compared with 8 cents for the same items by hand-pouring. In all cases the bitumen was spread by beginning at the lower elevation of the roadbed and working up grade.

After the bitumen had been applied the entire surface was covered with Granolithic stone ranging in size from ⅝ to ¼ in., and free from dust. The roadbed was then thoroughly rolled. If the bitumen appeared on the surface, more Granolithic was added, and the rolling was continued until the material ceased to creep and became a solid mass, showing an even, hard, dense, and granular surface. The rolling was generally begun at the gutter lines, and carried toward the crown, the shoulders first being rolled to prevent spreading. For several weeks after the road was open to traffic it was noticed that the bitumen appeared on the surface in places. Granolithic was placed in convenient piles at the sides of the road, and was spread on these places when required. In some cases it was found necessary or advantageous to do more rolling.

Two rollers were used on the work, one weighing 12 and the other 6 tons. The 12-ton roller was used for "spiking," for rolling the No. 2 course, and at times for rolling the No. 3 course.

It was found, however, that the 6-ton roller worked very satisfactorily for the No. 3 course, and also for the final rolling on the Granolithic.

Mr. Compton.

The cost of constructing 14 764 sq. yd. of roadway by this method was 72.95 cents per sq. yd., itemized as follows:

Labor	24.82	cents.
Stone	27.40	"
Rolling	4.84	"
Binder	13.74	"
Coal	0.95	"
Hauling binder	0.41	"
Incidentals, including supplies and repairs to plant	0.79	"
Total	72.95	cents.

The cost of rolling represents the hire of the 6-ton roller, at $10 per day, and the use of the Department's 12-ton roller, at $5 per day, which covers the salary of the engineer and watchman. The quantity of bituminous binder used was 1.9 gal. per sq. yd., which included a sealing coat on about 25% of the bituminous work. The bituminous binder flushed up very evenly on the greater part of the surface, so that the sealing coat was used only where it was deemed necessary.

The following rates of wages prevailed, per day of 8 hours:

Foreman	$3.50
Roller engineer	3.00
Labor	2.00
Carts	2.50
Double team	3.60
Watchman	2.00

The following prices were paid for materials:

Crushed stone, No. 2	$1.90	per cu. yd.
Crushed stone, No. 3	2.15	" " "
Crushed stone, dust	1.75	" " "
Crushed stone, Granolithic	2.15	" " "
Binder "Fairfield"	18.40	" ton of 2 000 lb.
Binder "Standard"	18.00	" " " 2 000 "
Coal	4.25	" "
Hauling binder	0.75	" "

"Gabbro" crushed stone was used throughout the entire work.

Except in a few places, the surface is in excellent shape at this time. The exceptions are in that portion of the work which was done during the latter part of September and the first part of October, when the nights and parts of the day were cool, thus preventing the bitumen from flushing to the surface in a satisfactory manner. This defect, it is believed, will correct itself in the spring when the warm

weather comes. The portion done during warmer weather has flushed up in a very satisfactory manner, and at present presents a bitulithic appearance. There are also at this time four or five weak places, each about 2 sq. ft. in area, due without doubt to the bitumen being too hot, which condition will occur at times at the bottom of the kettles. Mr. Compton.

WILLIAM H. CONNELL, ASSOC. M. AM. SOC. C. E.—As a part of the experiments on White Plains Avenue, Borough of the Bronx, several sections were built by the penetration method, using both asphaltic and tar binders. The description and detailed costs of the sections follow: Mr. Connell.

WHITE PLAINS AVENUE EXPERIMENTAL PAVEMENTS.

SECTION TWELVE: 29 + 00 to 31 + 00.

Bituminous Pavement, Penetration Method.

Foundation Course.—This consists of 1½-in. crushed trap rock and sufficient screenings to fill the voids. It was spread and rolled to a finished depth of 3½ in.

Surface Course or Paving Mixture.—Crushed trap rock (1½-in.) was spread and lightly rolled to a depth of 2¾ in. From 29 + 00 to 30 + 50 about 1½ gal. of Sanford and Strains Asphalt Binder was applied, and a layer of ¾-in. stone was spread and rolled. From 30 + 50 to 31 + 00, the Standard Oil Company's Special Binder was used.

Seal Coat.—The quantity applied was ¾ gal. per sq. yd., over which coarse sand and chips were spread and rolled. An 18-ton roller was used.

COST PER SQUARE YARD.

Base course	$0.329
Surface course	0.501
Seal coat	0.123
Hauling binder	0.006
Tar and stone heaters	0.003
Fuel	0.021
	$0.983

DETAILED COST PER SQUARE YARD.

Base 3½ in. thick.

		Per square yard.
65.7 cu. yd. trap rock, at $1.65	$108.40	$0.229
13.1 " " screenings, " 1.65	21.68	0.046
Labor		0.054
Total		$0.329

SURFACE COURSE.

		Per square yard.	
Labor	$93.56	$0.198	
1½-in. stone, 45.9 cu. yd., at $1.65	75.73	0.161	
¾-in. stone, 3.5 " " " 1.80	6.30	0.013	
Binder, 719 gal., at $0.085	61.11	0.129	= 1.52 gal. per sq. yd.
472 sq. yd., at $0.501	$236.70	$0.501	

Mr. Connell.

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	28	$4.00	$14.00
Pouring binder	1	16	2.25	4.50
Heating "	2	38	2.25	10.68
Carrying binder	3	34	2.25	9.56
Spreading top course 1½-in. stone	7	92	2.25	25.87
" ¾-in. stone	1	20	2.25	5.62
Wheeling stone	2	21	2.25	5.90
Heating stone	3	32	2.25	9.00
Rollerman	1	15	4.50	8.43
472 sq. yd., at $0.198				$93.56

SEAL COAT.

		Per square yard.
Labor	$21.31	$0.045
Chips, 3.5 cu. yd., at $1.90	6.65	0.014
Binder, 353 gal. " 0.085	30.01	0.064 = 0.75 gal. per sq. yd.
472 sq. yd. at $0.123	$57.97	$0.123

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	5	$4.00	$2.50
Pouring binder	1	5	2.25	1.40
Carrying "	3	12	2.25	3.37
Heating "	2	13	2.25	3.65
Spreading chips	1	7	2.25	1.96
Wheeling "	2	9	2.25	2.53
Heating "	2	11	2.25	3.09
Rollerman	1	5	4.50	2.81
				$21.31

The actual time required for laying the surface course (including the top course) was 3½ days, or an average of 135 sq. yd. per day. The ¾-in. stone required heating. Work was postponed three times on account of delayed shipments of binder, thereby increasing the cost. The actual time for laying the seal coat was 5 hours, or an average of 755 sq. yd. per day. The chips required heating.

SECTION ELEVEN: 27 + 97 to 29 + 00.

Barrett Manufacturing Company's "Modern Pavement."

Foundation Course.—This consists of 1½-in. crushed trap rock, spread and rolled to a finished depth of 3 in.

Surface Course or Paving Mixture.—A sufficient quantity of sand was spread to fill the voids. It was about ½ in. in thickness on top of the foundation course, over which about 1 gal. of Tarvia X per sq. yd. was applied. Then 1½-in. crushed trap rock was spread and

lightly rolled to a depth of 3 in. About 2 gal. of Tarvia X per sq. yd. was then applied by the penetration method. A layer of ¾-in. stone was spread and rolled. An 18-ton roller was used. Mr. Connell.

Seal Coat.—About 1 gal. per sq. yd. was applied, over which coarse sand was spread and rolled.

Cost per Square Yard.

Base course	$0.305
Surface course	0.698
Seal coat	0.219
Hauling binder	0.010
Tar heater	0.003
Fuel	0.021
	$1.256

Detailed Cost per Square Yard.

Base 3 in. thick.

		Per square yard.
33.8 cu. yd. trap rock, at $1.65	$55.77	$0.229
3.3 " " screenings, at 1.65	5.57	0.022
Labor	.....	0.054
		$0.305

Surface Course.

		Per square yard.
Labor	$59.06	$0.243
1½-in. stone, 19.6 cu. yd., at $1.65	32.34	0.133
Sand, 6.5 " " " 1.21	7.86	0.032
Binder 1st coat, 262 gal., " 0.095	24.89	0.102 = 1.08 gal. per sq. yd
Binder 2d coat, 480 gal., " 0.095	45.60	0.188 = 1.97 gal. per sq. yd.
243 sq. yd., at $0.698	$169.75	$0.698

Field Labor.

	No.	Hours.	Rate.	First Coat.
Foreman	1	6	$4.00	$3.00
Pouring	3	12	2.25	3.37
Heating binder	2	13	2.25	3.65
Spreading sand	10	20	2.25	5.62
Rollerman	1	2	4.50	1.12
243 sq. yd., at $0.069				$16.76

				Second Coat.
Foreman	1	16	$4.00	$8.00
Pouring	3	9	2.25	2.53
	2	16	2.25	4.50
Heating binder	3	9	2.25	2.53
	2	16	2.25	4.50
Spreading 1½-in. stone	10	20	2.25	5.62
	10	30	2.25	8.43
Rollerman	1	11	4.50	6.19
243 sq. yd., at $0.174				$42.30

Mr. Connell.

Seal Coat.

			Per square yard.
Labor		$17.32	$0.071
¾-in. stone,	5.0 cu. yd., at $1.80	9.00	0.037
Sand,	2.0 " " " 1.21	2.42	0.010
Screenings,	1.0 " " " 1.65	1.65	0.007
Binder,	240 gal. " 0.095	22.80	0.094 = 0.99 gal. per sq. yd.
243 sq. yd., at $0.219		$53.19	$0.219

Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	6	$4.00	$3.00
Heating binder	1	6	2.25	1.68
Pouring binder	2	10	2.25	2.81
Spreading sand	2	10	2.25	2.81
Spreading ¾-in. stone	2	16	2.25	4.50
Spreading screenings	3	3	2.25	0.84
Rollerman	1	3	4.50	1.68
				$17.32

The actual time for laying the surface course was: 1st coat, 6 hours, or an average of 324 sq. yd. per day; 2d coat, 8 hours, or an average of 243 sq. yd. per day.

The actual time for laying the seal coat was 6 hours, or an average of 324 sq. yd. per day.

Section Fifteen: 34 + 50 to 36 + 25.

Bituminous Pavement, Penetration Method.

Foundation Course.—This consists of 1½-in. crushed trap rock and screenings, spread and rolled to a finished depth of 3½ in.

Surface Course or Paving Mixture.—Crushed trap rock (1½-in.) was spread and lightly rolled to a depth of 2½ in. Tarvia—2.3 gal. per sq. yd.—was then applied, after which a layer of chips was spread and rolled.

Seal Coat.—About ¾ gal. of Tarvia per sq. yd. was applied, over which chips were spread and rolled with an 18-ton roller.

Cost per Square Yard.

Base course	$0.305
Surface course and seal coat	0.735
Fuel	0.021
Hauling binder	0.008
Tar heater	0.002
	$1.071

Detailed Cost per Square Yard.

Base 3½ in. thick.

		Per square yard.
57.4 cu. yd. trap rock, at $1.65	$94.71	$0.229
5.7 " " screenings, " 1.65	9.47	0.022
Labor		0.054
		$0.305

Mr. Connell.

Surface Course and Seal Coat Combined.

		Per square yard.
Labor	$100.77	$0.244
1½-in. stone, 40.2 cu. yd., at $1.65	66.33	0.161
Chips, 80 cu. yd., at $1.65	13.20	0.032
Binder, 1296 gal., at $0.095	123.12	0.298 = 3.14 gal. per sq. yd.
413 sq. yd., at $0.735	$303.42	$0.735

Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	48	$4.00	$24.00
Pouring and carrying tar	3	106	2.25	29.81
Heating tar	1	11	2.25	3.08
	2	48	2.25	13.50
Spreading 1½-in. stone	15	60	2.25	16.88
Spreading chips	1	16	2.25	4.50
Rollerman	1	16	4.50	9.00
				$100.77

The costs of the first coat of binder and the seal coat were not separated. About 0.75 gal. of binder per sq. yd. was used for a flush coat.

The actual time of laying was 5 days (4 full days and 2 part days), or an average of 80 sq. yd. per day. The length of time was due to poor supervision.

Section Sixteen: 36 + 25 to 37 + 75.

Bituminous Pavement, Penetration Method.

Foundation Course.—This consists of 1½-in. crushed trap rock and screenings, spread and rolled to a finished depth of 3½ in.

Surface Course or Paving Mixture.—Crushed trap rock (1½-in.) was spread and slightly rolled to a depth of 3 in., then 1½ gal. of Bermudez Road Asphalt per sq. yd. were applied, after which a layer of chips was spread and rolled.

Seal Coat.—Bermudez (½ gal. per sq. yd.) was applied, over which chips were spread and rolled, an 18-ton roller being used.

Cost per Square Yard.

Base course	$0.305
Surface course	0.519
Seal coat	0.123
Hauling binder	0.005
Tar heaters	0.002
Fuel	0 021
	$0.975

Mr. Connell.

Detailed Cost per Square Yard.

Base 3½ in. thick.

		Per square yard.
49.2 cu. yd. trap rock, at $1.65	$81.18	$0.229
4.9 " " screenings, at $1.65	8.12	0.022
Labor		0.054
		$0.305

Surface Course.

		Per square yard.	
Labor	$45.14	$0.128	
1½-in. stone, 34.5 cu. yd., at $1.65	56.92	0.161	
Chips 5.0 " " " 1.90	9.50	0.027	
Binder 550 gal., at 0.131	72.05	0.203	= 1.55 gal. per sq. yd.
354 sq. yd., at $0.519	$183.61	$0.519	

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	11	$4.00	$5.50
Pouring asphalt	2	18	2.25	5.06
Heating "	2	19	2.25	5.34
Carrying "	2	18	2.25	5.06
Spreading 1½-in. stone chips	5	36	2.25	10.12
	2	18	2.25	5.06
Rollerman	1	16	4.50	9.00
				$45.14

Seal Coat.

		Per square yard.	
Labor	$11.53	$0.032	
Chips, 5.0 cu. yd., at $1.90	9.50	0.027	
Binder, 173 gal., at $0.131	22.66	0.064	= 0.49 gal. per sq. yd.
354 sq. yd., at $0.123	$43.69	$0.123	

Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	4	$4.00	$2.00
Pouring asphalt	2	6	2.25	1.68
Squeegee	1	3	2.25	0.84
Heating asphalt	1	3	2.25	0.84
Carrying "	2	6	2.25	1.68
Spreading chips	2	6	2.25	1.68
	4	4	2.25	1.12
Rollerman	1	3	4.50	1.69
				$11.53

Nothing except the binder was heated on the ground. The chips were delivered hot from the plant, and remained warm until the end of the job.

The actual time of laying the surface course was 1 day and 3 hours, or an average of 257 sq. yd. per day. The actual time of laying the seal coat was 4 hours, or an average of 708 sq. yd. per day. Mr. Connell.

Section Seventeen: 37 + 75 to 39 + 22.

Bituminous Pavement, Penetration Method.

Surface and Base Course Spread in One Course.—This consists of 1½-in. crushed trap rock, spread and rolled to a depth of 6 in. Standard Oil Company's Binder "B" (1⅓ gal. per sq. yd.) was then applied, over which a layer of ¾-in. stone was spread and rolled.

Seal Coat.—Binder "B" (1.1 gal. per sq. yd.) was applied, after which chips were spread and rolled, an 18-ton roller being used.

Cost per Square Yard.

Surface and base course	$0.648
Seal coat	0.177
Fuel	0.021
Hauling binder	0.006
Tar heater	0.001
	$0.853

Detailed Cost per Square Yard.

Surface and Base Course.

		Per square yard.
Labor	$41.90	$0.121
1½-in. stone, 82.0 cu. yd., at $1.65.	135.30	0.390
¾-in. stone, 5.0 " " " 1.80.	9.00	0.026
Binder, 480 gal., at 0.08.	38.40	0.111 = 1.38 gal. per sq. yd.
347 sq. yd., at $0.648	$224.60	$0.648

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	18	$4.00	$9.00
Pouring and carrying oil	3	48	2.25	13.50
Heating oil	2	25	2.25	7.03
Spreading 1½-in. stone	10	20	2.25	5.62
" ¾ " "	2	16	2.25	4.50
Rollerman	1	4	4.50	2.25
347 sq. yd., at $0.121				$41.90

Seal Coat.

		Per square yard.
Labor	$25.12	$0.072
Chips, 3.0 cu. yd., at $1.90	5.70	0.017
Binder, 384 gal., at 0.08	30.72	0.088 = 1.11 gal. per sq. yd.
347 sq. yd., at $0.177	$61.54	$0.177

Mr. Connell.

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	12	$4.00	$6.00
Pouring and carrying oil	2	24	2.25	6.75
Heating oil	1	13	2.25	3.65
Spreading chips	2	23	2.25	6.47
Rollerman	1	4	4.50	2.25
				$25.12

The actual time of laying the surface course, excluding the base course, was 2¼ days, or an average of 154 sq. yd. per day. The actual time of laying the seal coat was 1½ days, or an average of 231 sq. yd. per day. Nothing but the binder was heated. The 1½-in. trap rock was spread in one course, without screenings.

TEMPERATURE, WEATHER, ETC.

Pavement.	Date.	Temperature.			Weather.
		9 A. M.	12 NOON.	3 P. M.	
Standard Oil Penetration	Oct. 18, '10. Begun.	59	65	66	Clear.
	Oct. 19, '10.	59	65	68	Clear.
	Oct. 21, '10.	55	57	59	Cloudy.
	Oct. 24, '10.	52	54	56	Clear.
	Oct. 25, '10. Finished.	55	60	60	Cloudy.
Tarvia Penetration	Oct. 13, '10. Begun.	48	52	55	Clear.
	Oct. 14, '10.	60	68	69	Fair.
	Oct. 15, '10.	61	63	..	Cloudy.
	Oct. 17, '10.	60	65	66	Cloudy.
	Oct. 18, '10.	59	65	66	Clear.
	Oct. 19, '10. Finished.	59	65	68	Clear.
Barber Penetration	Oct. 26, '10. Begun.	49	54	56	Clear.
	Oct. 27, '10.	58	65	67	Cloudy.
	Oct. 28, '10. Finished.	54	55	56	Clear.
"Modern Pavement"	Oct. 19, '10. Begun.	59	65	68	Clear.
	Oct. 21, '10.	55	57	59	Cloudy.
	Oct. 22, '10.	52	53	..	Rain.
	Oct. 24, '10.	52	54	56	Clear.
	Oct. 25, '10. Finished.	55	60	60	Cloudy.
Sanford and Strains (Penetration)	Nov. 14, '10. Begun.	38	44	43	Cloudy.
	Nov. 15, '10.	33	39	40	Cloudy.
	Nov. 17, '10.	38	40	42	Clear.
	Nov. 18, '10.	38	44	40	Clear.
	Nov. 22, '10. Finished.	37	42	46	Overcast.

Mr. Driscoll.

MICHAEL DRISCOLL, ESQ.*—Beacon Street, in the Town of Brookline, Mass., was re-surfaced in part with Tarvia, both by the penetration method and by what the Barrett Manufacturing Company calls the modern method. The speaker found that a flush coat of bitumi-

*Superintendent of Streets and Sewers, Brookline, Mass.

nous material was absolutely necessary for a perfect job. Where such a coat was used, however, it made the road very slippery; not in the sense that automobiles would skid on it, but so that horses with smooth shoes would skate over it. Many complaints of this condition were made by horsemen. By decreasing the crown of the road, the slipperiness can be avoided, to some extent, but where it still exists the only remedy seems to be to put on sand or stone screenings; in some cases the speaker used sand mixed with a heavy oil or with a tar preparation. The oil or tar prevents the sand from being blown off by high winds, and keeps the surface rough, but after a short time the sand wears out, and there is then more bituminous material on the surface than there was originally, with a corresponding increase in slipperiness. Mr. Driscoll.

Another part of this street, having a gradient of 5%, was re-surfaced with 3 in. of stone, using as a binder a preparation of Tarvia to which about 10 or 15% of asphalt was added. There was some apprehension that the road would be slippery on account of its grade, but, by omitting the flush coat, the surface remained rough enough to furnish good footing. This work cost 45 cents per sq. yd., including all labor, stone, and other material. For comparison it may be stated that the cost of ordinary water-bound macadam surfacing, from 3 to 4 in. thick, constructed in the same neighborhood under similar conditions, was 32 cents per sq. yd.

Malcolm H. Smith, Esq.—Where a Tarvia-bound macadam road had become slippery, the condition was remedied by applying an emulsion of oil and water and then covering the surface with screenings. The oil softened the binder as the screenings became bedded in the surface. Mr. Smith.

E. H. Rogers, M. Am. Soc. C. E.—In Newton, Mass., one section of bituminous road surfaced with tar by the penetration method is partly on a long 6% grade. As this section proved very slippery for horses, it was flushed with a thin layer of asphalt, with the result that a slightly elastic surface was formed which enabled the horses to obtain a good footing. Mr. Rogers.

Alexander F. Armstrong, M. Am. Soc. C. E.—Early in the spring of 1909, when the New York State Highway Department first began building highways, a number of contracts were in force for the construction of water-bound macadam highways. The Department realized that a different method of construction would have to be used in order to withstand the future traffic to which these highways would be subjected, and, consequently, the penetration method was adopted, because, with conditions existing at that time, it could be used by making supplementary agreements to the original contracts; it was evidently the cheapest; would require the least additional equipment on the part Mr. Armstrong.

Mr. Armstrong.

of the contractors; and also would not interfere with existing patents in regard to the use of bituminous materials in road construction.

During the past two years the State Highway Department has built approximately 673 miles of bituminous macadam highways by the penetration method and a little more than 2 miles by the mixing method. This work was done by contract, and in every case the bituminous material was sampled and tested rigidly to see that it met the requirements of the specifications under which it was used. On the whole, the roads built by the penetration method have been very satisfactory and the speaker could point to numerous cases where excellent bituminous macadam roads stretch for miles through the country. As to how long these roads will last in their present excellent condition, and as to the eventual cost of their maintenance and repairs, he is not prepared to state further than that, at present, both in regard to cost of maintenance and repairs, and their condition, they are all that could be desired. Of these 673 miles of road, a number have not been entirely satisfactory. The surface on one has raveled so badly during the cold, open weather of the present winter as to become nearly a complete failure, and will probaly have to be entirely re-surfaced. This is the only road surface of which the speaker knows which has been a complete failure. There have been other roads which have disintegrated in portions or in patches, on which repairs either have been made or will be necessary in the near future. In a number of cases this disintegration is known to be due to the brittleness of the bituminous materials, both tars and asphalts, and probably would have occurred to about the same extent if such materials had been applied by the mixing method.

When a bituminous macadam road surface gives satisfactory results, they are usually attributed to full compliance with the specifications; that is, that the materials and the workmanship were satisfactory, and also the weather and the traffic conditions over the road.

When a road surface fails, the tendency at first is to blame the bituminous binder, without ascertaining whether there are other causes which may be either entirely or partly responsible. In some cases it is easy to account for a failure; in others it is very difficult, as there are so many different things to be considered. In using bituminous materials by the penetration method, it is the speaker's belief that the kind of foundation, the drainage, the quality of the sand or screenings used as filler, the characteristics of the bituminous material used as binder, the workmanship, and the weather conditions, are all more or less responsible for the success or failure of the road; so that, when a surface is found to be disintegrating in places, it is far from fair to say immediately that the bituminous binder is entirely responsible for it, simply because it is the material which is at or near the surface of the road. A thorough investigation should be made, and the blame placed where it belongs.

Mr. Armstrong.

One of the objections claimed against the penetration method is that it is practically impossible to pour the bituminous material uniformly over the road. It is true that it cannot be poured with the same uniformity as could be secured in the mixing method, but, unless there is a high wind or very poor pouring, the distribution should be as uniform as required. A greater objection is that when the stone is spread on the road it frequently happens that it is not evenly distributed, and pockets of fine material appear in places. These pockets may be from 2 to 10 or 12 ft. in diameter, and the fine material may be either finer stones, screenings, sand, horse manure, or other foreign matter. Where one of these pockets of fine material exists and bituminous material is poured uniformly over the whole road, unequal conditions result. The bituminous material poured over a pocket does not penetrate to the same extent as on the average stretches of the road. Consequently, there results a fat spot, which, during warm weather, will either be soft or will bleed excessively, and during cold weather the bituminous material in these fat spots may become brittle enough to crack and disintegrate, when it may be brushed aside by the traffic and leave an unprotected and unbound patch which will disintegrate rapidly and grow in size because of the wear at its edges.

In building bituminous macadam roads by either method it is the speaker's belief that while short stretches of 100 or 500 ft. may indicate the wearing qualities of the materials and the class of construction used, such short stretches do not by any means give reliable data either as to what ultimately could be accomplished in the way of construction or the unit costs thereof. In some cases it is necessary for the Engineer or the Contractor to build a mile or more of road before things are running so smoothly as to secure a good road at a fair cost.

Cost data for 78 miles of 16-ft. bituminous macadam road built by the penetration method by the New York State Highway Department in 1909 showed that the average amount of filler used was 1 cu. yd. per 20 lin. ft. of surface; that the average amount of bituminous material poured was 1.6 gal. per sq. yd.; and that the average cost of all bituminous material and filler (of sand or screenings) in place complete in the top course, including cost of materials, labor, rolling, heating, etc., was 24 cents per sq. yd., made up as follows:

Material	$0.167 per sq. yd.
Labor	0.057 " " "
Equipment	0.010 " " "
Engineering	0.009 " " "
Total	$0.243 per sq. yd.

or about $2 250 per mile.

Mr. Armstrong.

The item for material is made up of approximately $0.144 for bituminous material and $0.023 for filler, fuel, etc. As about the same amount of filler, fuel, etc., would have been necessary on water-bound roads, and as the water sprinkling and the large amount of rolling necessary for water-bound roads was not required for bituminous macadam roads, some deduction should be made from the foregoing figures; so that the probable cost of the work (over and above that of building water-bound highways) was at the rate of about 20 cents per sq. yd., or about $1 900 per mile. As this cost is an average, based on 78 miles of highway built in various parts of the State, by various contractors, and under varying conditions, it may be considered as a fair estimate of the additional cost of bituminous macadam over that of water-bound macadam.

This additional cost of $1 900 per mile, however, does not take into consideration the fact that in many cases a local stone, which can be used satisfactorily with a bituminous binder, could not be used in a water-bound road. In some such cases the cost of importing a stone to secure a satisfactory water-bound road has more than offset the extra cost of the bituminous binder.

It is the speaker's opinion that a bituminous macadam highway could be built by the mixing method for nearly the same cost as by the penetration method, provided its length was sufficient to allow of using economically the proper apparatus. This length should not be less than 3 or 4 miles, and preferably somewhat longer.

Mr. Shirley.

H. S. Shirley,* Esq.—In the summer of 1907 the speaker constructed a small piece of road as an experiment in Towson, Md., using the penetration method. The road was re-surfaced with a blue gneiss to a depth of about 4 in., and thoroughly rolled until it was compact and hard. Over this surface was spread refined tar, using from 1¾ to 2 gal. per sq. yd. The tar was allowed to penetrate for about 3 hours, and then a covering of pea-sized gneiss was spread over the surface and thoroughly rolled, after which more screenings were added until the excess of tar was taken up. The following rates were paid for labor, for a 10-hour day: Laborers, $1.50; roller man, $2.50; double teams, $4.50; single teams, $3.00; foreman, $2.00. The cost of the work, in excess of the cost of water-bound macadam, was 33 cents per sq. yd. The section was very short, and naturally the cost was higher than if a greater length had been built. At present the road is in good condition, and not one cent has been spent on it for maintenance. It takes a traffic such as would be expected in a town of 2 500 people.

On another piece of road in Baltimore County, where the speaker used the penetration method, the old roadbed was scarified, and about 4 in. of trap rock were added and rolled down to about 3 in. Refined

* Roads Engineer, Baltimore County, Maryland.

tar was then applied (1½ gal. per sq. yd.), covered with stone chips, and rolled. More chips were added and rolled until the excess of tar was taken up, after which more tar (from ¼ to ½ gal. per sq. yd.) was spread. The surface was then covered with a layer of pea stone and rolled. This road cost 51 cents per sq. yd., including the cost of the stone, tar, and labor, and has been down about 2½ years with no additional expense for maintenance. Mr. Shirley.

In going into the penetration method on a larger scale, great difficulty was found in getting the tar from the tank cars. After trying drainage pumps, cellar pumps, and various other kinds, a rotary pump was purchased, and proved to be exceedingly satisfactory, as two men could load a 600-gal. tank wagon in 8 min.

If the haul from the station to the road was very long, there was a great deal of trouble in getting the tar to flow through the distributors, and, although the material had not lost its heat in the main body of the tank, it would harden in the orifices of the distributors. After several trials with different distributors, the following method was found to be as economical as with any of them, and more rapid. The tank wagon was fitted with a cut-off valve having a 3½-in. orifice. This valve was attended to by one man who filled a tub, placed under the orifice, with the bituminous material. Two men then carried the tub to two other men, who dipped out the material with ladles and spread it over the road surface. Two tubs were used, and while one was being emptied, the other was being filled ready for use. In this way a tank wagon could be unloaded in a very short time, in fact, 6 or 8 tank wagons could be unloaded in a day, although the haul from the tank car to the road was from 1 to 2 miles. By this method it was also found that a more uniform distribution of the tar was obtained.

Paul D. Sargent, Esq.*—About 20 000 sq. yd. of road were built in Maine, in 1910, under the speaker's direction, by the penetration method. The State supplied the bituminous binder, and the contractor furnished the stone and all the labor. The road was built in two courses, a first course of 4 in. and a top course of 2½ in., giving a total depth of 6½ in. of stone after rolling. The top course was treated by pouring on it 1½ gal. of bituminous material per sq. yd.; ¾-in. stone was then placed on that, and rolled in; then another coat of ½ gal. per sq. yd. was put on, and finally a layer of screenings. The total quantity of binder averaged 2.28 gal. per sq. yd. Tarite-asphalt, Tarvia X and Ugite No. 4, were the bituminous materials used, and the average cost was 10 cents per gal. delivered on the job; therefore, the bituminous binder cost about 23 cents per sq. yd. Mr. Sargent.

The specification for this piece of road was furnished by the United States Office of Public Roads, and required that the first course of stone be entirely filled with sand or screenings before laying the second

*State Highway Commissioner of Maine.

Mr. Sargent. course. This was done at the beginning of the work, but it left the top of the No. 1 course dusty, which prevented the second course from bonding to the first. After a few days, the filling of the bottom course was omitted. Probably it would have been better if the specification had been followed and the dust had been cleaned off the bottom course before the second course was laid.

Mr. Meeker. R. A. Meeker,* Esq.—The penetration method has been used in New Jersey with good, indifferent, and poor results, due both to the manner in which the pavement was laid and also to the materials used. Asphaltic oils from different parts of the country and several kinds of tar were used. Every carload of oil was tested, and, if shipped in barrels, one barrel from each carload was sampled. The tars were not tested. Although the bituminous material met the tests specified, this was no criterion of its value as a road binder. This was true particularly of fluxed asphalt, for if an asphaltic-base oil was used for a flux, the resulting product was good, but when a paraffin-base oil was used, the resulting product was very unsatisfactory.

It has been the speaker's experience that clean stone is of far more importance than dry stone. He has found that the bituminous material will adhere more rapidly to a clean and slightly moist stone than to a dry stone which is covered with dust, because the dust forms a film between the stone and the bitumen. There are times, of course, when the road may be too wet to permit bituminous work. The speaker has in mind one instance where a refined tar was applied to a road when it was quite wet, with the result that a failure occurred, owing to the formation of a film of steam on the stone. In applying bituminous materials to a road after a shower, he has found that bad results were obtained, and has come to the conclusion that, if the stone is thoroughly wet, it is advisable to stop using the bituminous material, whether it is a tar or an asphaltic product. If the stone is sprinkled with gasoline before the bituminous binder is applied, the latter will adhere to the cold stone, because the gasoline removes any dust which may be on the surfaces. This treatment is more expensive for the penetration than for the mixing method.

The stone used in constructing the roads is generally a trap rock. The bottom course is thoroughly filled and compacted with a 3-wheel, 10-ton roller. If a trap rock is used, the voids may be filled with either screenings or sand. With a limestone, the speaker prefers to use stone dust, because it has adhesive qualities of its own. His experience has been that unless the bottom course is filled, and most thoroughly and firmly compacted, the surface course will fail.

Some trouble was found in using a modified form of water sprinkler for applying the bituminous material. The material was unevenly distributed, because some of the holes became clogged. If the sprinkler

* State Road Supervisor of New Jersey.

had been provided with apparatus so that the material could have been kept hot, a better distribution would probably have been obtained. The American Tar Company's distributor has been used with very good results. The best results, however, were obtained by pouring the bituminous material by hand, or by applying it with a hose having a flat nozzle, the hose being attached to a tank in which the material was constantly heated. In constructing roads by the penetration method, unless the bituminous material is applied at a uniform temperature, the results will vary, no matter what the manner of application may be. Mr. Meeker.

In building some of the first bituminous roads in New Jersey, it was found that if a crown of $\frac{3}{4}$ or 1 in. per ft. was used, the surface would be slippery. The speaker believes that no road with a bituminous surface should have a crown of more than $\frac{1}{2}$ in. per ft., or, if the road is wide, $\frac{3}{8}$ in. per ft. is ample. If a proper crown is used, the objection of slipperiness can be prevented by immediately covering any bituminous application with a layer of stone screenings or sharp sand, and in this way a fairly gritty surface may be obtained. The amount of bituminous material generally used was $1\frac{1}{2}$ gal. per sq. yd. The speaker believes that, in the penetration method, the use of too much material is worse than not enough, because fairly good results may be obtained with a light application of the material, but if 3 or 4 gal. per sq. yd. are used, a wavy and sticky surface is likely to result, and make the traction heavy.

James C. Wonders,* Esq.—In Ohio there are about 20 000 miles of roads which have been improved under the charge of civil engineers and according to contracts and specifications prepared by them. Of this mileage, about two-thirds are gravel roads, and the greater part of the remainder is of broken stone. Less than 100 miles of bituminous roads have been built, and a few hundred miles are paved with vitrified brick. The use of brick for road construction was begun in Ohio about 25 years ago. The first bituminous road work was done in 1909. Four or five contracts of this nature were let by the State Highway Department, and, in addition, it put down an experimental road in which practically all the different products on the market were used. Although information regarding the lasting qualities of these bituminous roads is very much desired, their construction is of such recent date that but little has been gained. Mr. Wonders.

The work of building bituminous roads by county authorities has been encouraged very largely by the salesmen of these materials. It has been the experience of the speaker's department that, when the cost reaches $10 000 a mile, it becomes more economical to build a brick road than to build one of bituminous macadam.

* State Highway Commissioner of Ohio.

Mr. Wonders.

The roads with tar and asphalt binders are uniformly good at the present time. One difficulty which the speaker has noticed has caused serious criticism: If too much bituminous material is used, it is likely to exude and, especially if it is tar, it freezes at a moderate temperature and becomes slippery. Such a condition sometimes exists because the producers have been more interested in selling their material than in its economical use. The maximum quantity of binder used has been about 3 gal. per sq. yd. In order to remedy the slipperiness on such roads, a coating of torpedo sand or stone chips has been applied.

One thing that is to be desired is a test which will give an idea of the characteristics bituminous binders should have; and that test should not be burdened with too many technicalities. It is apparent that a tar should have a reasonably high melting point, so that it will not run out on the surface in warm weather; and it should have a reasonably low freezing point, so that it will not become brittle when the temperature is low. Although the recommendations of the Special Committee of this Society may be followed in the laboratory, and the qualities of the materials ascertained, the information desired is: Which of these qualities are essential for a first-class binding material?

An experience last year in building one of the bituminous roads near Columbus, Ohio, might be mentioned. In order that there should be no trouble about the delivery of the material, it was purchased from the largest company in this business in the United States, but, before the work was completed, word was received that the company was over-sold and could not supply the material. It is a question whether the available supply of tar and asphalt is sufficient, if the bituminous construction of highways is to be used extensively in the United States.

Mr. Crosby.

W. W. Crosby, M. Am. Soc. C. E. (by letter).—The various discussors seem to agree on the frequent convenience of using the penetration method, and on its successful results, in so far as a satisfactory determination of such results can be made at this early date. They also seem to agree that definiteness on this point can only be had after more time has elapsed for the actual use of such roads by traffic, and then by comparing the maintenance and first construction costs. The data furnished by the discussors concerning details of construction will be most valuable (if properly supplemented by maintenance and traffic data) after the lapse of a proper period, say, five years after construction.

From the various reports, the writer has received the impression that the practice in the penetration method is fairly uniform, varying slightly in detail according to local conditions, and with the inevitable "personal equation." There seems to be a general agreement as to the

importance of two points: first, cleanness, or freedom from dirt and "floury" material, in the stone aggregate; and, second, absence of excessive moisture in the aggregate at the time of applying the pitch. Mr. Crosby.

Mr. Armstrong refers to the difficulty of pouring bituminous materials uniformly—so frequently urged as a strong objection to the penetration method—but he very practically remarks that, although theoretical uniformity may be impossible, in practice it is easy to fulfill the requirements of this particular, and with this the writer agrees. Mr. Armstrong also refers to the difficulties of securing uniformity of the surface which is to receive the pitch. The desirability of this uniformity is apparent, and the writer has found that it can be obtained practically if the stone for the top course of macadam is not too small. It is also suggested that other advantages are likely to result from such use of larger stone.

There seems to be general agreement that, in building or rebuilding a macadam road by the penetration method (or, for that matter, with any method of using pitch in or on the macadam), it is advisable to reduce the cross-slope (or crown) to less than what has been formerly considered advisable for water-bound macadam.

In the more successful instances, the average quantity of pitch used, under the penetration method, seems to have been about 1½ gal. per sq. yd. The writer believes that it will be found possible to reduce this average safely in the future, especially in cases where the new macadam is less than 6 in. thick after rolling.

As usual, the reported costs per square yard are rather confusing, owing to the variations in the bases of estimating. The writer regrets the lack of uniformity in this matter. The figures given run from 10 to 60 cents per sq. yd. for the pitching of the macadam (in addition to what would have been the cost of water-bound macadam alone), or from 50 cents to $1.25 per sq. yd. as the total cost of the pitched macadam in place complete. A considerable reduction in these figures will probably be general in future work. Several discussors refer to the necessity for maintenance and expense on surfaces built under both the penetration and mixing methods, because of slipperiness, brittleness, dustiness, sweating or bleeding, etc., developed under traffic, but few give figures or other details of such after attention. The writer's figures on this point are given in Table 2, and, while regretting that figures for other roads do not seem to be available, presumes it is simply because the work reported on was almost wholly completed in the last working season (1910). It is probably unnecessary to dilate on the importance of such figures in the determination of the actual value of results, the first cost of which is recorded.

A number of interesting points are suggested by the discussion, such as methods for obviating excessive slipperiness with stone chips spread during warm periods; the tendency of some materials to sweat or

Mr. Crosby.

bleed excessively, and the remedies for this; the relief of tar brittleness secured by an after coat of asphaltic oil; the necessities, advantages, or disadvantages of a "sealing coat," etc., all of which will bear further attention with advantage.

One point, which, in the writer's opinion, is both interesting and important, is that referred to by Mr. Richardson on the possible advantage, obtained under the penetration method, of having an excess of pitch in the aggregate (over what is secured in the mixing method) for replenishing the surface as the latter "dries out" under traffic.

From this discussion, the writer has received the impression that many of the former objections to the use of the penetration method have been shown to be poorly founded, or at least to be more theoretical than practical, and that it has grown considerably in favor, perhaps at the expense of the mixing method. He believes also that the method of surface treatments (using the differentiations given in the title of this subdivision of the subject) has developed considerably in the past year, and is probably capable of much greater development, but perhaps at the expense of the penetration method. It is to be hoped that by cooperative effort during the next few years, the record of experiences, and the comparison of facts, definite information may be obtained concerning a great many of the points which are at present uncertain.

(7) THE USE OF BITUMINOUS MATERIALS BY MIXING METHODS.

By Messrs. Arthur H. Blanchard, F. C. Pillsbury, Frank J. Eppele, William H. Connell, R. A. Meeker, F. P. Smith, Clifford Richardson, J. W. Howard, and Paul D. Sargent.

Mr. Blanchard.

Arthur H. Blanchard, M. Am. Soc. C. E.—Bituminous pavements constructed by mixing methods may be considered under two types. Type A has as its characteristic a carefully-graded aggregate, and may be classified with sheet-asphalt and wood-block pavements, from the standpoint of adaptability under certain traffic conditions, while Type B, consisting of a pavement with an upper course containing a larger percentage of voids than Type A, is suitable for residential streets in cities, roads on State trunk lines, etc., which are subjected to excessive motor-car traffic and heavy commercial traffic, considered from the standpoint of the present traffic on trunk highways.

In the construction of bituminous pavements of Type A, the materials for the aggregate are broken stone and sand of graded sizes mixed so as to form a dense mixture, while the usual bituminous materials are natural asphalts, oil-asphalts, and refined tars, either alone or in combination with asphaltic materials. In connection with the construction, machines are used to heat, and, in some cases, to clean the aggregate, and thoroughly mix it with the bituminous binders. These plants are of two kinds, stationary and portable. The results secured by the use of pavements of this character have been very satisfactory, from the standpoint of durability, but in some instances the surfaces have been slippery and in other cases dusty. Particular attention should be given in the future to the nature of the material used as a flush coat, as it is possible to construct a pavement of this type so that the surface will not be either slippery or dusty.

Pavements of Type B are usually constructed by using as an aggregate either the run of the crusher or a mixture of certain sizes of broken stone, with or without sand, and a bituminous cement of natural asphalt, oil-asphalt, heavy asphaltic oil, tar, or a combination of tar and asphaltic materials. The two methods which have been adopted in constructing pavements of this type are, first, the hand-mixing method, in connection with which unheated stone is generally used, and second, by mixing the aggregate and the bituminous material with mechanical mixers. The results—to date—by these methods in some cases have proved satisfactory and in others both insufficient and uneconomical. The results have generally proved satisfactory when the method and the materials were adaptable to local conditions, when the roads were well constructed with a bituminous cement of good quality,

Mr. Blanchard.

and the work was in charge of a competent inspector. Here, again, the function of the flush coat and the course of mixed aggregate has not been fully appreciated. In the future much attention should be devoted to the design of a mixer, including a dryer as an essential adjunct under certain conditions. One type of plant of this character should be designed so as to be economically adaptable to road work on which portable crushers are used, the output of which is necessarily limited. Distributors for the application of a flush coat should also receive attention.

In Rhode Island bituminous pavements have been constructed by the mixing method since 1906. Judging from the results obtained to date the most economical and efficient pavements for heavy combination trunk line traffic under normal conditions in Rhode Island have been those constructed with tar in the mix and an oil-asphalt flush coat, the oil-asphalt which has proved most satisfactory during all seasons of the year being a Texaco asphalt having a melting point of from 90° to 95° cent., as determined by the test proposed by the Special Committee of this Society. The first pavement of this type was built in 1908 in East Providence.* When inspected, in January, 1911, the surface was in an excellent state of preservation and practically dustless, the flush coat of asphalt being in such condition as to take the prints of the calks of horses' hoofs, although the air temperature was 0° cent. The average traffic for 12 hours on this road during the summer of 1909 was 355 horse-drawn vehicles and 810 motor vehicles. Under normal conditions, when constructed under competent supervision, and in the months of June to September, inclusive, a pavement of the class referred to will cost from 25 to 35 cents per sq. yd. more than ordinary macadam.

For roads subjected to traffic consisting of 90% motor cars and a light horse-drawn vehicle traffic, the results in Rhode Island indicate that a flush coat on a bituminous pavement constructed by the mixing method is not necessary. The excellent condition, when examined in October, 1910, of the 1906 section in Charlestown built of Providence tar, and the 1908 section in Narragansett built of Tarvia, are evidence that a road of this type is economical and efficacious. The summer traffic during 1909 on the Narragansett road averaged 42 horse-drawn vehicles and 410 motor vehicles for a period of 12 hours. A road of this character can be constructed, under normal conditions, during the months from June to September, inclusive, with refined tar or an oil-asphalt for from 15 to 25 cents per sq. yd. in excess of the cost of ordinary macadam, provided the construction work is supervised by a competent inspector. The estimates given are based on 1910 labor and material cost data.

*The construction of this section of the East Providence road is described in *Transactions*, Am. Soc. C. E., Vol. LXI, p. 445 *et seq.*

Mr. Blanchard.

The 1909 experimental sections built in Barrington, while the speaker was in charge of bituminous work in Rhode Island, have been open to traffic for more than a year. The sections were all built by the mixing method, each section having a paint coat of the same kind of bituminous material as was used in the mix. The materials were Malden Tarite, Springfield Tarite, Tarite-Asphalt E (20% asphalt), Tarite-Asphalt J (10% asphalt), Texaco Macadam Binder and the U. G. I. Road Compound No. 4. When examined in January, 1911, the various sections exhibited the following characteristics: The Malden Tarite and Springfield Tarite sections presented an excellent, close-grained, mosaic surface, the surface coat on the top of the stones having entirely disappeared. The surface afforded an excellent foothold for horses. There has been a slight abrasion of the road metal composing the upper surface. Very little difference was noted between the Tarite-Asphalt E and Tarite-Asphalt J sections, except that the mosaic surface of the stones was more apparent in the latter than in the former. The surface presented a fairly good foothold for horses, and was practically dustless. On the section constructed with Texaco Macadam Binder the stones composing the upper layer were not visible. The flush coat was remarkably dustless, and, at a temperature of 0° cent., was slightly impressed by the calks of horses' hoofs. On the surface constructed with the U. G. I. Road Compound No. 4 the stones were not visible. The surface was extremely hard and smooth, and hence slippery, but was dustless.

The road on which these various sections were constructed is between West Barrington and Nayatt Point, in the Town of Barrington. Each section was built on a tangent, and the grades varied from 0.50 to 1.14 per cent. The sub-grade encountered was composed of a very hard layer of clay and sand over a foundation bed of bricks, which had been placed there several years before by the town highway authorities. Almost the entire length of the road was through a wooded district, the shade being very dense. Previous to reconstruction, the road had been surfaced with gravel. In June and July, 1909, the average traffic for 8 hours was as follows:

Horse-drawn vehicle traffic.	One-horse vehicles	71
	Two-horse vehicles	3
Motor vehicle traffic.	Motor cycles	1
	Motor runabouts	9
	Motor touring cars (four or five seats)	43
	Motor touring cars (six or seven seats) including limousines and landaulets	17

Mr. Blanchard.

The contract was let originally for ordinary macadam construction, and later a secondary agreement was drawn up by which the extra labor required on the bituminous work was paid for as per agreement covering prices for different grades of labor plus a commission of 15 per cent. The prices paid to laborers on bituminous work were $2.00 per day of 10 hours for experienced men, and $1.75 per day for ordinary laborers. One-horse teams and drivers were paid $3.50, and two-horse teams and drivers $5.00 per day.

The following plant equipment was used: A Kelly-Springfield 15-ton roller; a tar-coating machine manufactured by the American Tar Company, the rent of which was $2.00 per day. The contractor supplied three tar kettles, mixing platforms, square-pointed shovels, pails, dippers, and rakes, at a rent of $0.25 per kettle and accessories per day. Brooms, potato-hooks, and steel-bodied wheel-barrows were supplied by the State.

The macadam was built in two courses: The foundation course consisted of broken stone varying from 1½ to 2¾ in. in longest dimensions, which was 6 in. deep when loose, and was rolled to 4 in. The top course consisted of stone of the same quality, varying in size from ¾ to 1½ in., which was 3 in. deep when loose, and was rolled to 2 in. The rock used was a chlorite gneiss, having the following characteristics:

Determinations.	Results.
Specific gravity	2.80
Weight per cubic foot	175 lb.
Water absorbed per cubic foot	0.46 lb.
Percentage of wear	4.6
French coefficient of wear	8.6
Hardness	15.7
Toughness	8
Cementing value	Fair.

The width of macadam was 14 ft. and the crown throughout was ¾ in. per ft. The top-course stone was mixed by hand with bituminous material until all stones were thoroughly coated. Most of the mixing was done by hand, with long-handled, square-pointed shovels, on plank platforms composed of three sections, each 8 ft. square. The tar-coating machine was used on the sections built with Springfield-Tarite and the U. G. I. No. 4 Road Compound. This machine and the method of using it was described by the speaker in a paper on "Use of Binding Materials in the Construction of Metalled Roads."* The mixing gang consisted of three experienced and five common laborers. The flush-coat gang consisted of one experienced and two common laborers. In applying the flush-coat house brooms were used, the material being supplied to the spreader in steel-bodied wheel-barrows.

* Presented before the Second International Road Congress at Brussels, in August, 1910.

Mr. Blanchard.

Table 1 contains general information and Table 2 cost data relative to the various experimental sections; Table 3 gives analyses of the bituminous materials used. The analyses were made in accordance with the methods proposed by the Special Committee of this Society appointed "To Report on Bituminous Materials for Road Construction, and on Standards for their Test and Use."

The following notes, relative to details of construction are of interest; they were taken from the records of the Resident Engineer, Irving W. Patterson, Jun. Am. Soc. C. E.

Malden Tarite.—At the prevailing temperature, the material refused to run from the bungs of the barrels with sufficient rapidity to keep the work constantly progressing. For this reason the heads of the barrels were knocked out and the contents emptied into the kettles in a very short time. On this section rolling was done each day until a thorough compacting had been effected and the material had set up so that there was no appreciable movement under the wheels of the roller. Rolling was never done until the material had been down over night. The first day rolling was done about 9 A. M., before the material had softened excessively. Later, as the material began to set up, rolling was delayed until the middle of the day because in the earlier hours the stiffness of the surface did not allow of compacting. The surface had set up in such a manner that painting might have been done at the end of about 6 days.

Texaco Macadam Binder.—The weather conditions during the construction of this section were the most unfavorable during the entire period of construction. Owing to frequent rains and almost constant mists, it was impossible to keep the stone dry. Work was carried on, however, except when rain was actually falling, because of the lateness of the season and the desire to finish to a certain point before weather conditions prevented further bituminous construction. The stone was mixed with no great difficulty when dry, but when damp, as was generally the case, it was very difficult to get it coated. The material was exceedingly slow in setting up. It was impossible to apply the flush coat until 3 weeks after laying the stone. Even at that time the surface felt very elastic under foot pressure.

Tarite-Asphalt (10%).—The heads of the barrels had to be removed in order to allow the material to flow into the kettles. Mixing was difficult, because of the low temperature and the consequent stiffness of the bituminous materials. Owing to the rapidity with which the material hardened on contact with the cold stones, it was necessary to mix very small batches at one time. Rolling on this section was done in the middle of the day. Rolling was also done on the day the material was laid because if the surface was left unrolled over night permanent compacting was made very difficult. At the end of 2 days the surface had set up sufficiently to allow painting.

TABLE 1.—General Information.

Material.	Length, in feet.	Average grade, percentage.	Air Temperature, in Degrees, Fahrenheit.		Date mixing began.	Date mixing ended.	Average temperature of material, in degrees, Fahrenheit.	Bituminous material in mix, in gallons per square yard.	Bituminous material in flush coat, in gallons per square yard.	Daily progress, in square yards (mix).	Daily progress, in square yards (paint).
			Average.	Min.							
Malden Tarite	350	0.50	80	60	Sept. 20.	Sept. 22.	240	1½		232	550
Texaco Macadam Binder	280	2.12	80	65	Sept. 22.	Sept. 29.	255	1½		217	400
Tarite-Asphalt (20%)	168	0.28	70	55	Sept. 29.	Sept. 30.	255	2		216	400
Tarite-Asphalt (10%)	169	0.83	65	50	Sept. 30.	Oct. 2.	250	2		200	400
Springfield Tarite	291	1.14	75	50	Oct. 2.	Oct. 6.	240	2		150	500
U. G. I. Road Compound No. 4.	275	1.14	75	50	Oct. 6.	Oct. 9.	290	2		155	450

TABLE 2.—Cost Data.

Material.	Material per gallon, f. o. b.	Freight, per gallon.	Loading and hauling, per gallon.	Material on road, per gallon.	Heating material, per gallon.	Mixing material, per gallon.	Painting, per gallon.	Total cost of material in place, per square yard.*
Malden Tarite	$0.080	$0.010	$0.007	$0.097	$0.008	$0.029	$0.021	$0.297
Texaco Macadam Binder	0.110	Price per gallon includes freight.	0.005	0.115	0.008	0.030	0.021	0.357
Tarite-Asphalt (20%)	0.098	0.009	0.004	0.111	0.008	0.026	0.033	0.383
Tarite Asphalt (10%)	0.092	0.009	0.004	0.106	0.008	0.028	0.036	0.376
Springfield Tarite	0.080	0.009	0.004	0.093	0.007	0.043	0.032	0.369
U. G. I. Road Compound No. 4.	0.070	0.025	0.003	0.099	0.007	0.044	0.037	0.391

* The total cost per square yard given in Table 2 is the cost over and above the cost of ordinary macadam. The cost of a 2-in. course of ordinary macadam was 16 cents per sq. yd.

Mr. Blanchard.

TABLE 3.—Analyses of Bituminous Materials.

Mr. Blanchard

	Malden Tarite.	Texaco Macadam Binder.	Tarite-Asphalt (20%).	Tarite-Asphalt (10%).	Springfield Tarite.	U. G. I. Road Compound No. 4.
Specific gravity	1.222	0.965	1.176	1.211	1.244	1.167
Water soluble, organic	0.4	0.3	0.5	0.6	0.2	0.3
Water soluble, inorganic	0.4	0.0	0.1	0.0	0.4	0.1
Free carbon	25.0	0.3	18.6	22.5	24.5	2.4
Ash	0.1	0.0	0.0	0.1	0.0	0.1
Solubility in cold CCl_3	.....	99.6	74.0	69.5	.....	.....
Fixed carbon	29.4	8.0	27.4	26.2	30.7	22.1
Paraffine	.....	0.2	.....	.....	.....	.....
Melting point, original material	.....	.....	.....	.....	.....	.....
Evaporation in 5 hours at 170° cent	15.0	16.6	10.5	13.5	11.9	15.5
Melting point residue	62	65	70	67	63	74
Penetration residue, 4° cent	2	46	2	1	3	0.5
Penetration residue, 25° cent	15	105	12	10	15	3
Evaporation in 5 hours at 205° cent	.....	19.1	17.5	18.0	.....	.....
Melting point residue	.....	116	80	85	.....	.....
Penetration residue, 4° cent	.....	29	1	1	.....	.....
Penetration residue, 25° cent	.....	58	2	2	.....	.....
Solubility 88° Baumé naphtha	.....	77	.....	.....	.....	.....
Character of solution, sticky or oily	.....	sticky	.....	.....	.....	.....
Distillation to 300° cent	19.7	.....	.....	18.7	20.7	16.8
Up to 105° cent	0.0	.....	.....	0.3	0.0	0.0
105° to 170° cent	0.4	.....	.....	1.0	0.3	0.0
170° to 225° cent	7.8	.....	.....	3.3	4.8	0.0
225° to 270° cent	8.3	.....	.....	9.4	10.9	10.4
270° to 300° cent	3.2	.....	.....	4.7	4.7	6.4

Tarite-Asphalt (20%).—The quantity of material used, the manner of rolling, and the length of time necessary for setting up varied very slightly from the experience with Tarite-Asphalt (10%), but there was somewhat more difficulty in removing the material from the barrels.

Springfield Tarite.—Rolling was done on this section in the warm part of the day, because the season was rather late, and the mornings and evenings being cold, the material was stiffened so much that rolling did not compact the surface well. The flush-coat might have been applied on the third day.

U. G. I. No. 4 Road Compound.—The first section laid with this material was allowed to remain over night before rolling, but this section was never compacted as well as where the rolling followed close after the laying of the surface. Rolling in the early morning appeared to affect the surface in no way except to break up the stones of the No. 2 course. Rolling in the middle of the day, except on the day on which the material was laid, appeared to be but slightly more effective than rolling in the early morning. Hence it was found absolutely necessary to roll very soon after laying the No. 2 course. The great change produced by relatively slight changes of temperature can be judged from the fact that in the early morning at this season of the

Mr. Blanchard. year the material in the barrels might be split to pieces with an axe, while earlier in the season it would flow slowly from the bungs.

Based on experience acquired while in charge of bituminous work in Rhode Island, and investigations conducted in the United States and Europe, the speaker considers that many of the failures of bituminous pavements constructed by mixing methods have been due to one or more of the following factors: Poor bituminous binder; bituminous material not adapted to the method used; the use of an insufficient quantity of bituminous material; poor mix; an aggregate unsatisfactory in quality; injudicious selection of sizes of road metal in the aggregate; aggregate covered with dust or dirt; damp broken stone or sand; overheated broken stone, sand, or bituminous material; the use of excessively heavy three-wheel rollers; the coating of the bituminous-covered aggregate with roadside dust, either before rolling or preceding the application of the flush-coat; insufficient bond between the foundation course of macadam and the bituminous course, due in certain cases to over-compacting the lower course; and, finally, incompetent supervision of the handling of materials and of the details of construction.

Mr. Pillsbury. F. C. Pillsbury, M. Am. Soc. C. E.—Road surfaces should be designed, as nearly as possible, for the traffic to be sustained, and, at the present time, it seems that it is possible to draw the line closely between the necessity for a mixed bituminous macadam and for a less expensive type on the one hand, and a superior pavement on the other.

There is no doubt that it is possible to construct on a suitable base a mixed bituminous macadam which will sustain extremely heavy horse-drawn vehicle traffic, and it is even possible that in the future a pavement may be found which will sustain the very heaviest horse-drawn vehicle traffic. On the other hand, there is no doubt that even the most successful bituminous macadam constructed by penetration or layer methods will not sustain a traffic equal in weight to that which may be borne by the mixed composition.

This seems to prove the superiority of a mixed aggregate over other kinds of bituminous macadam when heavy loads drawn by horses must be sustained. For medium traffic, partly or even almost altogether automobile, it seems that the penetration and layer methods of construction will answer. They are less expensive, of course, but if the mixing method is superior in one case, why should it not be in the other? It is probable that its superiority will give much greater permanence, and that this may prove it to be more economical. On the other hand, a large volume of swiftly-moving automobile traffic may disturb the actual surface, causing abrasion to such an extent that either mixed, penetration, or layer treatment will be badly disturbed on the surface after two or three years, so that it would be economy in any case to restore the surface continually before destruction by surface applications of some form of bitumen.

Mr. Pillsbury.

The speaker does not intend these remarks to be an argument in favor of one method of construction over another; they are simply made for the purpose of calling attention to the fact that it is of first importance to consider whether it is economical to construct bituminous macadam by the mixing method instead of laying the most expensive pavements, such as vitrified brick, wood block, sheet-asphalt, first-class granite, etc., or instead of cheaper types of bituminous surfaces.

The speaker believes that there are many cases where an asphaltic bituminous macadam could be laid which would answer all the purposes now served by brick or wood block, except that wood block is quieter. Bituminous macadam, however, is quieter than brick or granite-block pavements. The cost of such pavement should not exceed one-half that of the brick or other more expensive forms mentioned above. Asphaltic bituminous macadam must be constructed only of selected materials placed with the greatest care.

The actual operation of mixing and placing an aggregate may be accomplished at a low cost by careless methods, but the results will be uncertain, and, without doubt, will result in failures in many cases. So much depends on the personal factor, that skilled and experienced labor should be employed. This leads to a question of machinery *versus* hand labor. From experience the speaker has no doubt that machine work is more satisfactory, as machinery may be used which will mix uniformly and thoroughly. It is almost impossible to obtain a uniform product by hand mixing.

One of the most interesting roads, on which both hand and machine mixing were used, was constructed in the summer of 1910 on the State highway between the City of Lynn and the Town of Revere, in Massachusetts. This road carries all the business traffic between Boston and Lynn and other cities and towns north and east, as well as a large volume of automobile travel.

If the road had not been subject to very heavy horse-drawn vehicle traffic, a mixed bituminous macadam would not have been constructed. The traffic was so heavy that the Massachusetts Highway Commission thought it advisable to use a bituminous material which was, as far as known, least likely to oxidize and lose its plastic and adhesive qualities until after the greatest period of wear had been obtained. It was finally decided to use a composition of natural oil and refined natural asphalt, and to mix this in a thorough manner with an excellent quality of trap rock.

The work was complicated and made difficult and expensive by the necessity of keeping the road open to travel. The entire width of the roadway was only 22 ft., hence only 11 ft. were available at one time.

At first the mixing was done by hand, but the cool nights and mornings made it difficult to combine the stiff bituminous material with the cold stone satisfactorily, and it was found that the work was progressing too slowly. Hence, it was decided to mix by machinery.

Mr. Pillsbury. Two Smith concrete mixers were used. The stone was heated and the mixing accomplished with entire satisfaction. The two Smith mixers turned out daily sufficient material to cover about 800 sq. yd. of surface. This amount kept three steam rollers busy, and was all that could possibly be handled.

This practical type of surface and the method of operation is to be strongly recommended for roads which are somewhat remote from a central mixing plant and are subjected to heavy traffic.

Mr. Eppele. FRANK J. EPPELE, ESQ.* (by letter).—In Mercer County, New Jersey, four pieces of road have been re-surfaced to a depth of 1½ in. or more after compression with materials called by some special trade name, where the mineral aggregate was coated mechanically with a bituminous binder. These roads are giving by far the best and most satisfactory results.

This work has been done at a cost of about 70 cents per sq. yd., under a guaranty of 5 years, and, after 2 years of service, does not show any appreciable indication of wear; in fact, judging by present surface conditions, these roads should last for a period of 15 years without any material expense. The writer understands, however, that similar work would cost at the present time from 80 to 90 cents per sq. yd., due to increases in the cost of labor and materials, and the question of royalties to be paid under patents granted for the construction of a bituminous road having 21% or less of voids.

A mineral aggregate of the desired proportions as to size, coated mechanically with the required amount of first-class bitumen of the proper consistency, will, without doubt, give an ideal wearing surface for new as well as re-surfaced roads, but, under existing circumstances, and in accord with a recent legal decision, it is practically impossible to compile a specification, under which bids could be received with unlimited competition, without infringement on certain patent rights. On the other hand, to ask for bids under a specification calling for a restricted form of construction, or any special kind of pavement, would immediately meet with disfavor in the eyes of the public, and raise the cry of favoritism, regardless of the merits involved. Without open competition, it is useless to expect the cost of this form of construction to be brought within the limit of reasonable expenditure for general highway improvement, and so its use will probably be restricted to districts of fixed realty value and population. If this is true, we are forced to eliminate, when considering methods of bituminous construction, the mechanical coating of the mineral aggregate having a low percentage of voids, and to confine our thoughts practically to the spreading of the road metal, the application of the asphaltic binder to the surface, by hand or otherwise, and then to depend on its penetration qualities to give a variable condition of coated work.

*County Engineer, Mercer County, New Jersey.

Mr. Eppele.

Being dependent, then, on the penetration method to produce the form of construction for future highway improvement which will be within an allowable range of cost, as well as a method of re-surfacing roads already constructed, the main question seems to be one of proper asphaltic binder to be used. On this question it is the writer's judgment that a fluxed natural asphalt will give the best and most lasting results, and, in the end, prove to be the cheapest bitumen to use, even if the initial cost is somewhat above that of the artificial bitumens now on the market.

Mr. Connell.

WILLIAM H. CONNELL, ASSOC. M. AM. SOC. C. E.—The following discussion is confined to a general description of the experimental sections on White Plains Road which were built in order that a comparison might be made as to the cost and durability of bituminous pavements laid by the mixing and by penetration methods, with water-bound macadam and other types of pavements.

The White Plains Road experimental pavements consist of eighteen sections, ranging in length from 100 to 350 ft., with a width of 21 ft. Fourteen of these sections comprise the bituminous pavement group, two of which consist of a wearing surface of broken stone and sand with an asphaltic binder, built by the mixing method and laid on a concrete foundation. The wearing surfaces of twelve sections were laid on a broken stone foundation, five built by the penetration and seven by the mixing method. In all but one section built by the mixing method the stone was heated. The remaining four are made up as follows: Hassam Portland Cement Concrete Pavement, water-bound macadam, Sicilian Asphalt Company's Asphalto, which in appearance resembles ground Kentucky Rock Asphalt, and a surface application of asphalt binder and sand on an old macadam strip. In all but one section, where paving gravel was used in the wearing surface, the stone, both for foundation and wearing surface, was Clinton Point crushed trap rock.

The section built with the surface application and those built by the penetration methods are discussed under Topics Nos. 5 and 6, respectively.

These experiments were made with a view to determine by a service test a suitable medium-priced pavement for suburban residential streets, and also a cheaper grade of pavement for country roads and parkways, where the constantly increasing automobile traffic has made it imperative to lay something better than a water-bound macadam pavement. Therefore, it is to be hoped that the information obtained through observation of these experiments will be of value, both from engineering and economic points of view. The maintenance cost of water-bound macadam subjected to automobile traffic is appalling, and consequently it will be necessary in the future to

Mr. Connell.

construct a pavement where, even though the initial cost may be a little higher, the maintenance cost will be very much lower.

Accurate cost data of the construction of the various sections have been kept, and cost data of the maintenance will be also kept. Analyses of the bituminous binders have been made by the United States Office of Public Roads, in accordance with their methods of testing bituminous binders, and in accordance with the recommendations of the Special Committee of the American Society of Civil Engineers on "Bituminous Materials for Road Construction," and levels have been taken which will make it possible to ascertain the amount of wear at any time.

It is the intention to take a traffic census of the road once each month up to December, 1911. The census will be taken on Fridays, Saturdays, Sundays, and Mondays, in order to take into account two holidays and two normal week days. Due to the fact that the speaker considers that these sections should undergo a year's use before any very definite knowledge as to their relative merits can be expressed, he does not desire to report on their present condition.

The White Plains Road is a wide thoroughfare extending northeastward through the northerly section of the Borough of the Bronx to Mount Vernon, and on toward White Plains. The experimental roadway is laid on the east side of the road, and extends from the intersection of White Plains Avenue and the Boston Road to Burke Street, a distance of 4 120 ft. The maximum grade is 1% and the crown is $\frac{3}{8}$ in. to 1 ft. No streets intersect White Plains Road between the Boston Road and Burke Street, therefore all the sections will be subjected to the same amount of traffic. A double-track electric railway runs along White Plains Avenue, and the pavement is carried up to the outside girder rail of the track.

The water-bound macadam section was laid as a basis of comparison, both with regard to wear and cost of construction. The cost of experimental work of this character, where the sections are short and all differ in methods of construction, is necessarily high, as the organization can hardly be perfected in making satisfactory progress before the section is completed. Therefore it may be safe to assume that the excess cost of the water-bound macadam may be used to determine the percentage to be deducted from the actual cost of any other section, in order to ascertain its normal cost. The total cost per square yard of the ordinary macadam, 6 in. deep, as laid on this road, was $0.771, which is about 25% in excess of the average cost.

Work on the experimental road was begun on September 29th, and was completed on November 22d, 1910. The number of sections under construction at the same time ranged from two to nine. It will be observed, from the lateness of the season and daily record of the temperature taken during the period of construction, that the work was

not done under the most favorable conditions. The temperature when the work was in progress varied from 40 to 79° Fahr., the average being 57° Fahr. However, it was thought better to make a start and finish this season than wait about 9 months for more perfect conditions. Mr. Connell.

It is to be observed that the labor item is somewhat high. This is partly due to the rate of pay, namely, $2.25 for an 8-hour day, and partly to the fact that the style of work was entirely new to the foreman as well as to the men. There are several points which stand out in connection with the work, which it may be worth while noting, namely: the comparatively low cost of Section 5, where the binder was mixed with cold stone in a concrete mixer; the high cost per square yard for fuel where the stone was heated; the additional cost per square yard for mixing tar and asphalt; the excess of binder used in the seal coat of several sections; the high cost of handling the Amiesite pavement, due entirely to the paving mixture being stiff, owing to cold weather; the difference in the cost of the concrete foundations laid by the mixing method and that by the process of spreading cement and screenings mixed, sprinkling and rolling, as in water-bound macadam; and the high cost of labor, namely, $0.054 per sq. yd., for spreading and rolling the broken stone foundation course, due to re-handling the material.

The methods of construction used on White Plains Road, both for penetration work and the mixing method, were rather crude, and unless a suitable pressure machine is used for penetration work which will insure an equal distribution of bituminous binder, and unless an economical mixer is used for heating and mixing the stone and bituminous binder, thereby decreasing the labor cost, the cost of work of this character will be unnecessarily high in comparison with water-bound macadam. The method of construction, therefore, should be taken into consideration in comparing the relative cost of the various sections with the water-bound macadam.

A description of the methods of construction, the binders used, and cost data relative to the various sections constructed by the mixing method, follows. A traffic census taken in January, 1911, and a table of temperatures, are also appended.

White Plains Avenue Experimental Pavements.

Section One: 0 + 00 to 3 + 25.

Bituminous Pavement, Mixing Method, laid by the Barber Asphalt Company.

Foundation Course.—A concrete foundation was laid, to a finished depth of 4½ in., in the proportions of 1:3:6.

Surface Course or Paving Mixture.—The paving mixture consists of the following mineral aggregate by weight, 73% paving gravel, 19½% sand, 1½% dust, and 5.4% Bermudez Asphalt, 55° penetration. The

Mr. Connell.

mixing was done at the Barber Asphalt Plant, then laid at a temperature of 220° and rolled to a finished depth of 1¾ in. with a 10-ton roller.

Seal Coat.—From 0 + 00 to 1 + 52, ¼ gal. of Asphaltic Cement per sq. yd. was applied, over which sand was spread.

COST PER SQUARE YARD.

Concrete foundation	$0.639
Surface course	0.715
Seal coat	0.085
	$1.439

DETAILED COST PER SQUARE YARD.

Concrete Base, 4½ in. thick.

		Per square yard.
Labor	$168.00	$0.219
1½-in. stone, 86 cu. yd., at $1.65	141.90	0.185
Sand, 43 " " " 1.21	52.03	0.068
Cement, 95 bbl., " 1.35	128.25	0.167
767 sq. yd., at $0.639	$490.18	$0.639

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	24	$5.00	$15.00
Laborers	22	528	1.75	115.50
Watchmen	2	48	1.75	10.50
Timekeeper	1	24	3.00	9.00
Team	1	24	6.00	18.00
				$168.00

SURFACE COURSE.

The cost was about 71½ cents per sq. yd., estimated from the cost of Bronx 55° penetration, deducting $27 for the difference in the cost of gravel and trap rock.

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	8	$5.00	$5.00
Laborers	12	96	1.75	21.00
Watchman	1	8	1.75	1.75
Tampers	4	32	2.50	10.00
Rakers	3	24	2.50	7.50
Smoothers	2	16	2.00	4.00
Rollerman	1	8	4.50	4.50
Timekeeper	1	8	3.00	3.00
767 sq. yd., at $0.074				$56.75

SEAL COAT.

		Per square yard.
Labor	$9.22	$0.026
Chips, 5 cu. yd., at $1.90	9.50	0.027
Binder, 89 gal., at $0.131	11.66	0.032 = 0.248 gal. per sq. yd.
359 sq. yd., at $0.085	$30.38	$0.085

Mr. Connell.

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	4	$5.00	$2.50
Pouring asphalt	2	6	2.25	1.68
Squeegee	1	3	2.25	0.84
Heating asphalt	1	3	2.25	0.84
Carrying asphalt	2	6	2.25	1.68
Spreading chips	2	6	2.25	1.68
				$9.22

The actual time of laying the surface course was 8 hours, or an average of 767 sq. yd. per day. The actual time of laying the seal coat was 3 hours, or an average of 957 sq. yd. per day.

SECTION TWO: 3 + 25 to 6 + 50.

Bituminous Pavement constructed by the Mixing Method and laid in accordance with Borough of The Bronx Specifications.

Foundation Course.—This consists of 1½-in. crushed trap rock, thoroughly rolled with an 18-ton roller to a finished depth of about 3¾ in.

Surface Course or Paving Mixture.—The paving mixture is composed of two parts of crushed trap rock passing through a 1¼-in. ring and not containing more than 5% of dust, one part of sand and 7.4% by weight of Bermudez Asphalt, 55° penetration. The mixing was done at an asphalt plant, and the paving mixture was laid at a temperature of 220° and rolled to a finished depth of 2½ in. with a 10-ton roller.

Seal Coat.—From 3 + 25 to 4 + 78, ¼ gal. of Asphaltic Cement per sq. yd. was applied, over which chips were spread and rolled.

COST PER SQUARE YARD.

Base course	$0.283
Surface course	0.751
Seal coat	0.081
	$1.115

DETAILED COST PER SQUARE YARD.

		Per square yard.
106.4 cu. yd. trap rock, at $1.65	$175.56	$0.229
Labor	.....	0.054
		$0.283

SURFACE COURSE.

		Per square yard.	
Mixing at plant	$110.48	$0.144	
Labor	51.25	0.067	
Hauling	93.84	0.122	
Asphalt cement, 1 600 gal., at $0.131	209.60	0.273 = 2.09 gal. per sq. yd.	
¾-in. stone... 54 cu. yd. " 1.25	67.50	0.088	
Sand..... 30½ " " " 0.75	22.87	0.030	
Dust.....2 760 lb. " 3.50	4.83	0.006	
Grit..... 3 cu. yd. " 1.25	3.75	0.005	
Coal.....8 030 lb. " 3.00	12.05	0.016	
767 sq. yd., at $0.751	$576.17	$0.751	

Mr. Connell.

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	8	$5.00	$5.00
Laborers	8	64	1.75	14.00
Watchmen	3	24	1.75	5.25
Tampers	4	32	2.50	10.00
Rakers	3	24	2.50	7.50
Smoothers	1	8	2.00	2.00
Rollerman	1	8	4.50	4.50
Timekeeper	1	8	3.00	3.00
767 sq. yd., at $0.067				$51.25

SEAL COAT.

		Per square yard.
Labor	$9.52	$0.026
Chips 3 cu. yd., at $1.90	5.70	0.016
Asphalt cement .95 gal. " 0.13	12.35	0.035 = 0.26 gal. per sq. yd.
Wood ¼ cord " 6.50	1.52	0.004
361 sq. yd., at $0.081	$29.19	$0.081

LABOR.

	No.	Hours.	Rate.	Totals.
Heating asphalt	1	11	$1.75	$2.38
Pouring "	1	11	1.75	2.38
Spreading chips	2	22	1.75	4.76
				$9.52

The actual time of laying the surface course was 8 hours, or an average of 767 sq. yd. per day.

SECTION THREE: 6 + 50 to 8 + 00.

Bituminous Pavement, Mixing Method.

Foundation Course.—A concrete base was laid, under the supervision of Mr. C. I. Williams, of Utica, N. Y., to a finished depth of 4¾ in.; 1½-in. stone was spread to a depth of 6 in., then screenings and cement mixed dry in proportions of 3:1 were spread, rolled, and sprinkled with water. The operation was repeated until the voids were filled. An 18-ton roller was used.

Paving Mixture or Surface Course.—The paving mixture is composed of 2 parts of crushed trap rock passing through a 1¾-in. ring and not containing more than 5% of dust, 1 part of sand, and 8% of Bermudez asphalt, 55° penetration. The mixing was done at an asphalt plant, and the paving mixture was laid at a temperature of 220° and rolled to a finished depth of 2 in. with a 10-ton roller.

Seal Coat.—The surface course was painted with about ¼ gal. of Asphaltic Cement per sq. yd., over which chips were spread and rolled.

Mr. Connell.

Cost per Square Yard.

Concrete foundation	$0.502
Surface course	0.90
Seal coat	0.08
	$1.482

Detailed Cost per Square Yard.

Concrete Base, 4¾ to 5 in. thick.

			Per square yard.
Labor		$19.96	$0.056
1½-in. stone	59 cu. yd., at $1.65	97.35	0.275
Screenings	9.0 " " " 1.65	14.85	0.042
Cement	20¼ bbl. " 1.35	27.34	0.077
354 sq. yd. at	$0.45	$159.50	$0.45
Spreading 1½-in. stone	0.052		
Total	$0.502		

Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	9	$4.00	$4.50
Laborers	5	43	2.25	12.09
Rollerman	1	6	4.50	3.37
				$19.96

Surface Course.

		Cost per square yard.
Mixing at plant	$105.14	$0.297
Labor in field	32.06	0.090
Hauling	40.80	0.115
Asphalt cement, 742.5 gal., at $0.13	96.52	0.273 = 2.1 gal. per sq. yd.
Trap rock	22.5 cu. yd., at 1.25	28.12	0.079
Sand	9.0 " " 0.75	6.75	0.020
Dust	1 440 lb., " 3.50	2.52	0.008
Coal, soft, 5 300 lb.	" 3.00	7.95	0.022
" hard, 400 "	" 6.00	1.20	0.003
354 sq. yd., at $0.907		$321.06	$0.907

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	6	$5.00	$3.75
Laborers	8	48	1.75	10.50
Watchman	1	6	1.75	1.31
Tampers	2	12	2.50	3.75
Rakers	3	18	2.50	5.62
Smoothers	1	6	2.00	1.50
Rollerman	1	6	4.50	3.38
Timekeeper	1	6	3.00	2.25
354 sq. yd., at $0.090				$32.06

Mr. Connell.

SEAL COAT.

			Per square yard.
Labor		$8.80	$0.025
Chips	3 cu. yd., at $1.90	5.70	0.016
Asphalt cement, 82.5 gal.,	" 0.13	10.72	0.030 = 0.23 gal. per sq. yd.
Wood, ½ cord	" 6.50	3.25	0.010
354 sq. yd., at $0.080		$28.47	$0.081

LABOR.

	No.	Hours.	Rate.	Totals.
Heating asphalt	1	6	$1.75	$1.31
Pouring asphalt	2	12	1.75	2.62
Spreading chips	2	12	1.75	2.62
Timekeeper	1	6	3.00	2.25
				$8.80

The actual time of laying the surface course was 5 hours, or an average of 454 sq. yd. per day.

SECTION FOUR: 8 + 00 to 11 + 25.

Bituminous Pavement, Mixing Method, laid under the supervision of the Standard Oil Company.

Foundation Course.—This consists of 1½-in. crushed trap rock thoroughly rolled with an 18-ton roller to a finished depth of about 3¾ in.

Surface Course or Paving Mixture.—The paving mixture consists of the following mineral aggregate by weight: 58% of ¾-in. crushed trap rock, 20% of ⅜-in. chips, 15% of sand, and 7% of Standard Oil Company's Special Asphalt Binder. The binder, stone, and sand were heated on the ground and mixed by hand on mixing boards; the mixture was spread and rolled to a finished depth of 2¾ in. with an 18-ton roller.

Seal Coat.—From 9 + 95 to 10 + 66, ⅓ gal. per sq. yd. was applied, over which chips were spread and rolled.

COST PER SQUARE YARD.

Base course	$0.283
Surface course	0.964
Hauling binder	0.009
Fuel	0.063
Asphalt and stone heaters	0.004
	$1.323

DETAILED COST PER SQUARE YARD.

Base, 3¾ in. thick.

			Per square yard.
106.4 cu. yd. trap rock, at $1.65		$175.56	$0.229
Labor			0.054
			$0.283

Mr. Connell.

Surface Course.

		Per square yard.
Labor	$368.14	$0.480
¾-in. stone, 60 cu. yd., at $1.80	108.00	0.140
Chips, 18 cu. yd., at $1.90	34.20	0.045
Sand, 12 cu. yd., at $1.21	14.52	0.019
Binder, 2 688 gal., at $0.08	215.04	0.280 = 3.5 gal. per sq. yd.
767 sq. yd., at $0.964	$739.90	$0.964

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	24	$4.00	$12.00
	2	48	4.00	24.00
Spreading	1	24	2.25	6.75
	2	48	2.25	13.50
Mixing	8	192	2.25	54.00
	4	108	2.25	30.37
Wheeling	6	280	2.25	78.75
Heating and carrying oil	4	96	2.25	27.00
	5	120	2.25	33.75
Heating stone	5	235	2.25	66.09
Rollermen	3	39	4.50	21.93
767 sq. yd., at $0.480				$368.14

During the first 3 days one gang worked; during the last 3 days two gangs worked. The total quantity laid was 767 sq. yd., or an average of 85 sq. yd. per day per gang.

Section Five: 11 + 25 to 14 + 50.

Bituminous Pavement, Mixing Method, laid under the supervision of the Texas Company.

Foundation Course.—This consists of 1½-in. crushed trap rock, thoroughly rolled with an 18-ton roller to a finished depth of about 3¾ in.

Surface Course or Paving Mixture.—The paving mixture consists of ¾-in. trap rock and 6.7% by weight of Texas Macadam Binder. The binder was heated and mixed with the cold stone in a standard concrete mixer on the ground. The paving mixture was spread, and rolled with an 18-ton roller to a finished depth of 2¾ in.

Seal Coat.—From 12 + 36 to 13 + 03, a seal coat of Texas 55 Special (about 1¼ gal. per sq. yd.) was applied, over which chips were spread and rolled.

Cost per Square Yard.

Base course	$0.283
Surface course	0.721
Tar heater	0.001
Fuel	0.021
Hauling binder	0.006
	$1.032

Mr. Connell.

Detailed Cost per Square Yard.

Base, 3¾ in. thick.

		Per square yard.
108.6 cu. yd. trap rock, at $1.65	$179.19	$0.229
Labor	.....	0.054
		$0.283

Surface Course.

		Per square yard.
Labor	$182.86	$0.234
¾-in. stone, 81 cu. yd., at $1.80.	145.80	0.186
Binder, 1 728 gal., at $0.12	207.36	0.265 = 2.2 gal. per sq. yd.
Concrete mixer (rent), 7 days, at $4.00	28.00	0.036
782 sq. yd., at $0.721	$564.02	$0.721

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	44	$4.00	$22.00
Spreading	2	90	2.25	25.31
Feeding mixer	4	171	2.25	48.09
Wheeling	3	132	2.25	37.12
	3	25	2.25	7.08
Heating binder	1	48	2.25	13.50
Rollerman	1	9	4.50	5.06
Mixer engineer	1	44	4.50	24.75
782 sq. yd., at $0.234				$182.86

The actual time of laying the surface course was 5 days and 4 hours, or an average of 142 sq. yd. per day.

Section Six: 14 + 50 to 17 + 75.

Amiesite, laid under the supervision of the Amiesite Company.

Foundation Course.—This consists of 1½-in. crushed trap rock and sufficient screenings to fill the voids. This was spread, and then rolled with an 18-ton roller to a finished depth of 3½ in.

Surface Course or Paving Mixture.—The paving mixture was shipped from the Amiesite plant, and laid in two courses: the first course was composed of 1½-in. stone with binder, etc.; laid and rolled to a finished depth of 2 in. The second or wearing course was composed of ⅝-in. stone, binder, etc., laid and rolled to a finished depth of 1 in. The paving mixture, therefore, had a finished depth of 3 in. An 18-ton roller was used.

Cost per Square Yard.

Base course	$0.329
Surface Course—	
Labor, spreading	0.155
" unloading	0.132
Hauling	0.179
	$0.795*

*Does not include cost of Amiesite.

Mr. Connell.

Detailed Cost per Square Yard.

Base, 3½ in. thick.

		Per square yard.
106.4 cu. yd. trap rock, at $1.65	$175.56	$0.229
21.3 " " screenings, at $1.65	35.11	0.046
Labor	.....	0.054
		$0.329

Surface Course.

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	21	$4.00	$10.50
Laborers	17	356	2.25	100.12
Rollerman	1	15	4.50	8.43
766 sq. yd., at $0.155				$119.05
Screenings, 1½ cu. yd., at $1.65		$2.47	$0.003 per sq. yd.	

Labor Unloading 72 Cu. Yd. Amiesite from Railroad Cars.

	No.	Hours.	Rate.	Totals.
Foreman	1	34	$4.00	$17.00
Laborers	9	297	2.25	83.90
766 sq. yd., at $0.132				$100.90

Hauling from Railroad Depot.

	No.	Hours.	Rate.	Totals.
Teams	3	15	$5.00	$9.38
	7	56	5.00	35.00
	9	27	5.00	16.88
	6	21	5.00	13.12
	8	52	5.00	32.50
	3	12	5.00	7.50
	3	24	5.00	15.00
	4	13	5.00	8.12
766 sq. yd., at $0.179				$137.50

The actual time of laying the surface course was 2 days and 4 hours, or an average of 306 sq. yd. per day. The material was very hard, due to the length of time it remained in the cars, making the work of unloading from the cars and laying the surface very difficult. Under favorable conditions the labor should be reduced by 70 per cent.

Mr. Connell.

Section Seven: 17 + 75 to 20 + 67.

Sicilian Asphalt, laid by the Sicilian Asphalt Company.

Foundation Course.—This consists of 1½-in. crushed trap rock, rolled with an 18-ton roller to a finished depth of 4½ in.

Surface Course or Paving Mixture.—The paving mixture is called "Asphalto," and was mixed at the Sicilian Asphalt Company's plant, hauled to White Plains Avenue and spread. The paving mixture was not rolled until comparatively cool, and then a Sicilian Asphalt 10-ton grooved roller was used. The finished depth was 1½ in. A light layer of cement was then sprinkled over the surface.

Cost per Square Yard.

Base course	$0.382
Surface course—	
Field labor	0.030
Cement	0.001
Hauling	0.087
	$0.500*

Detailed Cost per Square Yard.

Base, 4½ in. thick.

		Per square yard.
129.9 cu. yd. trap rock, at $1.65	$214.33	$0.299
13.0 " " screenings, at $1.65	21.40	0.029
Labor		0.054
		$0.382

Surface Course.

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	5	$5.00	$3.12
Spreading	10	50	1.75	10.93
Rakers	2	10	2.50	3.12
Spreading cement	1	6	1.75	1.31
Rollerman	1	6	4.50	3.37

718 sq. yd., at $0.030			$21.85
Cement, 2½ bags, at $1.35 per bbl	$0.840	$0.001 per sq. yd.	

Hauling.

19 loads, at $3.25	$61.75	$0.087 per sq. yd.

The actual time of laying the surface course was 5 hours, or an average of 1 148 sq. yd. per day.

Section Eight: 20 + 67 to 22 + 75.

Bituminous Pavement, Mixing Method, laid under the supervision of the Barrett Manufacturing Company.

Foundation Course.—This consists of 1½-in. crushed trap rock and screenings, rolled with an 18-ton roller to a finished depth of 3¾ in.

* Cost of Asphalto not included.

Mr. Connell.

Surface Course or Paving Mixture.—The paving mixture consists of 3 parts of ¾-in. crushed trap rock, 1 part of chips and 6½% by weight of Tarvia X. The stone and Tarvia X were heated on the ground and mixed by hand on mixing boards, spread, and rolled with an 18-ton roller to a finished depth of 2¾ in.

Seal Coat.—About 0.6 gal. per sq. yd. of Tarvia X was applied, over which chips were spread and rolled.

Cost per Square Yard.

Base course	$0.305
Surface course	0.913
Seal coat	0.137
Hauling binder	0.009
Tar and stone heaters	0.002
Fuel	0.063
	$1.429

Detailed Cost per Square Yard.

Base, 3¾ in. thick.

		Per square yard.
68.1 cu. yd. trap rock, at $1.65	$112.36	$0.229
6.8 " " screenings, at $1.65	11.23	0.022
Labor	.....	0.054
		$0.305

Surface Course.

		Per square yard.
Labor	$206.60	$0.421
1½ in. stone, 6 cu. yd., at $1.65	9.90	0.177
¾-in. " 43 " " 1.80	77.40	
Chips, 14 " " 1.90	26.60	0.054
Binder, 1 344 gal., at 0.095	127.68	0.261 = 2.7 gal. per sq. yd.
491 sq. yd., at $0.913	$448.18	$0.913

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	42	$4.00	$21.00
Spreading	1	42	2.25	11.81
Mixing	4	169	2.25	47.53
Wheeling	3	142	2.25	39.93
Heating binder	2	94	2.25	26.43
Heating stone	3	150	2.25	42.19
	4	29	2.25	8.15
Rollerman	1	17	4.50	9.56
491 sq. yd., at $0.421				$206.60

Seal Coat.

		Per square yard.
Labor	$33.24	$0.068
Chips, 3.5 cu. yd., at $1.90	6.65	0.014
Binder, 288 gal., at $0.095	27.36	0.055 = 0.58 gal. per sq. yd.
491 sq. yd., at $0.137	$67.25	$0.137

Mr. Connell.

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	8	$4.00	$4.00
Pouring binder	1	8	2.25	2.25
Squeegee	1	8	2.25	2.25
Heating and carrying binder	3	26	2.25	7.31
Spreading chips	2	16	2.25	4.50
Wheeling and heating chips	4	32	2.25	9.00
Rollerman	1	7	4.50	3.93
				$33.24

The actual time of laying the surface course was 6 days and 2 hours, or an average of 78½ sq. yd. per day. The actual time of laying the seal coat was 1 day, or an average of 491 sq. yd. per day. The chips required heating, as they were wet and cold.

SECTION NINE: 22 + 75 to 24 + 50.

Bituminous Pavements, Mixing Method.

Foundation Course.—This consists of 1½-in. crushed trap rock and screenings, spread and rolled to a finished depth of 3¾ in.

Surface Course or Paving Mixture.—From 22 + 75 to 23 + 02, the binder consists of a mixture of tar and 10% of refined Bermudez asphalt. From 23 + 02 to 24 + 00, the binder consists of a mixture of tar and 15% of Bermudez asphalt of 60° penetration. From 24 + 00 to 24 + 40, the binder consists of a mixture of tar and 15% of Bermudez apshalt of 110° penetration. From 24 + 40 to 24 + 50, Bermudez asphalt of 110° penetration was used for a binder. The paving mixture consists of 3 parts of ¾-in. stone, 1 part of chips, and 7% by weight of bitumen. The stone and binder were heated on the ground, mixed by hand on mixing boards, spread, and rolled with an 18-ton roller to a finished depth of 2¾ in.

Seal Coat.—About 0.6 gal. of asphalt per sq. yd. was applied, over which chips were spread and rolled.

COST PER SQUARE YARD.

Base course	$0.305
Surface course	1.034
Seal coat	0.146
Fuel	0.063
Hauling binder	0.009
Tar and stone heaters	0.002
Mixing binder	0.076
	$1.635

DETAILED COST PER SQUARE YARD.

Base, 3¾ in. thick.

		Per square yard.
57.3 cu. yd. trap rock, at $1.65	$94.54	$0.229
5.7 " " screenings, " 1.65	9.45	0.022
Labor	.....	0.054
		$0.305

Mr. Connell.

Surface Course.

		Per square yard.
Labor	$197.84	$0.479
1½-in. stone, 5 cu. yd., at $1.65	8.25	0.181
¾-in. " 37 " " " 1.80	66.60	
Chips, 12 " " " 1.90	22.80	0.055
Binder, 1 115 gal., " 0.118.	131.57	0.319 = 2.7 gal. per sq. yd.
413 sq. yd., at $1.034	$427.06	$1.034

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	47	$4.00	$23.50
Spreading	1	46	2.25	12.93
Mixing	4	184	2.25	51.75
Wheeling	3	146	2.25	41.06
Heating binder	4	192	2.25	53.98
	3	18	2.25	5.06
Rollerman	1	17	4.50	9.56
413 sq. yd., at $0.479				$197.84

Seal Coat.

		Per square yard.
Labor	$23.50	$0.056
Chips, 3 cu. yd., at $1.90	5.70	0.014
Binder, 264 gal., at $0.118	31.15	0.076 = 0.64 gal. per sq.yd.
413 sq. yd., at $0.146	$60.35	$0.146

Labor, Mixing Tar and Asphalt on Ground.

Foreman	3 hours	$1.50
Laborers	2 men, 9 hours	2.53
"	3 " 24 "	6.75
288 gal., at $0.037		$10.78

This labor included part of two days, requiring heating up twice and emptying from kettles to barrels for future use. The material should be mixed as used, requiring only an extra kettle kept hot, with asphalt in it ready to mix.

Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	6	$4.00	$3.00
Pouring	1	6	2.25	1.68
Squeegee	1	6	2.25	1.68
Heating and carrying binders	3	19	2.25	5.34
Spreading chips	2	12	2.25	3.37
Wheeling and heating chips	4	20	2.25	5.62
Rollerman	1	5	4.50	2.81
Total				$23.50
Mixing binder at plant, $31.56 = $0.076 per sq. yd.				

Mr. Connell.

The cost of this section was high because the work was not continuous, due to waiting for binder and also due to cold weather, which made the stone harder to heat. The chips were wet and cold, and required heating.

The actual time of laying the surface course was 6 days, or an average of 69 sq. yd. per day. The actual time of laying the seal coat was 6 hours, or an average of 551 sq. yd. per day.

Section Ten: 24 + 50 to 27 + 97.

Bituminous Pavement, Mixing Method.

Foundation Course.—This consists of 1½-in. crushed trap rock and screenings, spread and rolled to a finished depth of 3¾ in.

Surface Course or Paving Mixture.—The paving mixture consists of 3 parts of ¾-in. trap rock, 1 part of ⅜-in. trap rock chips, and 6% by weight of refined tar. The binder and stone were heated on the ground, and mixed by hand on mixing boards, spread, and rolled with an 18-ton roller to a finished depth of 2¾ in. The binder used was refined tar made up in accordance with specifications drawn up by the Office of Public Roads, of the United States Department of Agriculture.

Seal Coat.—About 0.6 gal. per sq. yd. was applied, over which chips were spread and rolled.

Cost per Square Yard.

Base course	$0.305
Surface course	0.868
Seal coat	0.135
Fuel	0.063
Hauling binder	0.008
Tar and stone heaters	0.003
	$1.382

Detailed Cost per Square Yard.

Base, 3¾ in. thick.

		Per square yard.
113.6 cu. yd. trap rock, at $1.65	$187.44	$0.229
11.3 " " screenings, " 1.65	18.74	0.022
Labor		0.054
		$0.305

Surface Course.

		Per square yard.
Labor	$308.91	$0.377
¾-in. stone, 82.5 cu. yd., at $1.80	148.50	0.181
Chips, 27.5 cu. yd., at $1.90	52.25	0.064
Binder, 2 016 gal., at $0.10	201.60	0.246 = 2.46 gal. per sq. yd.
819 sq. yd., at $0.868	$711.26	$0.868

Mr. Connell.

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	52	$4.00	$26.00
Spreading	1	20	2.25	5.62
	2	63	2.25	17.71
Mixing	4	201	2.25	56.53
Wheeling stone	4	99	2.25	27.84
	5	140	2.25	39.37
Heating stone	3	36	2.25	10.12
	5	200	2.25	56.25
Heating tar	5	176	2.25	49.50
	3	23	2.25	6.47
Rollerman	1	24	4.50	13.50
819 sq. yd., at $0.868				$308.91

Seal Coat.

		Per square yard.
Labor	$50.89	$0.062
Chips, 6.0 cu. yd., at $1.90	11.40	0.014
Binder, 480 gal., " 0.10	48.00	0.059 = 0.59 gal. per sq. yd.
819 sq. yd., at $0.135	$110.29	$0.135

Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	9	$4.00	$1.50
Boring	1	10	2.25	2.81
Squeegee	1	10	2.25	2.81
Heating tar	6	54	2.25	15.18
Carrying tar	4	35	2.25	9.84
Spreading chips	1	9	2.25	2.53
Wheeling chips	4	31	2.25	8.72
Rollerman	1	8	4.50	4.50
819 sq. yd., at $0.062				$50.89

The actual time of laying the surface course was 6 days and 4 hours, or an average of 126 sq. yd. per day. The actual time of laying the seal coat was 1 day and 2 hours, or an average of 655 sq. yd. per day. The chips were not heated.

Cost of Base Course, Fuel, Hauling Binder, Tar Kettles, Stone Heaters, Etc.

3¾-in. Macadam Base.		Per square yard.
379.5 cu. yd. 1½-in. trap rock, at $1.65	$626.17	$0.229
37.9 " " screenings, " 1.65	62.61	0.022
Labor		0.054
2 733 sq. yd		$0.305

Mr. Connell.

3¾-in. Macadam Base.		Per square yard.
221.3 cu. yd. 1½-in. trap rock, at $1.65	$365.14	$0.229
44.2 " " screenings, " 1.65	73.02	0.046
Labor		0.054
1 592 sq. yd		$0.329

3¾-in. Macadam Base.		Per square yard.
321.4 cu. yd. 1½-in. trap rock, at $4.165	$530.31	$0.229
Labor		0.054
2 316 sq. yd		$0.283

4½-in. Macadam Base.		Per square yard.
129.9 cu. yd. 1½-in. trap rock, at $1.65	$214.33	$0.299
13.0 " " screenings, " 1.65	21.43	0.029
Labor		0.054
718 sq. yd		$0.382

4¾-in. Macadam Base.		Per square yard.
59 cu. yd. 1½-in. trap rock, at $1.65	$97.35	$0.275
Labor		0.054
354 sq. yd		$0.329

Spreading Base Course.

Foreman, 98 hours, at 50 cents	$49.00
Laborers, 1 149 " " 28⅛ "	323.15
Rollerman, 82 " " 56¼ "	46.12
7 713 sq. yd., at $0.054	$418.27

Fuel.

41 cords, at $6.50 ..... $266.50
$0.048 per sq. yd. on 5 568 sq. yd.
Estimated: $0.021 per sq. yd. for penetration and mixing with cold stone.
" $0.063 " " " " " mixing with hot stone.

The heating surface required to heat the stone was twice the area of that required to heat the tar and asphalt, therefore the mixing jobs, when the stone was heated, required three times the quantity of wood used on penetration or cold stone sections, when the binder only was heated.

Hauling Binder.—The cost of hauling 12 000 gal. was $93.70 = $0.0078 per gal., the cost for the same haul should be: 5 loads = 500 gal. × 5 = 2 500 gal. per day, at $6.50 = $0.0026 per gal.

Tar Kettles and Stone Heaters.—Tar kettles cost about $100; their life is 10 years. Interest at 4% for 10 years, plus sinking fund = $8.006 per year for 10 years = $120.06. The total cost is $120.06 for 10 years = $12.006 per year; 120 working days = 10 cents per day.

Stone heaters of flat plates cost about $15.00; their life is 10 years; interest at 4% for ten years, plus sinking fund = $1.20 per year for 10 years = $18.00. Total cost $1.80 per year; 120 working days = 1½ cents per day.

Mr. Connell.

Table of Temperatures.

Pavement.	Date.	Temperature.			Weather.
		9 A. M.	12 NOON.	3 P. M.	
Barber Asphalt.....	Sept. 29, '10. { Begun ... / Finished . }	62	65	67	Clear.
Bronx Asphalt, 55.....	Sept. 30, '10. Begun.....	62	65	67	Hazy.
	Oct. 1, '10. Finished.....	66	70	..	Clear.
Bronx Asphalt.....	Oct. 14, '10. Begun	60	68	69	Fair.
	Oct. 15, '10. Finished....	61	63	..	Cloudy.
S. O. Mixture.....	Nov. 9, '10. Begun.....	39	45	46	Cloudy.
	Nov. 10, '10	48	54	55	Clear.
	Nov. 11, '10	44	48	43	Cloudy.
	Nov. 12, '10	35	37	..	Cloudy.
	Nov. 14, '10	38	44	43	Cloudy.
	Nov. 15, '10. Finished ...	33	39	40	Cloudy.
Texas Oil Mix.....	Oct. 5, '10. Begun	71	74	76	Cloudy.
	Oct. 6, '10	72	76	79	Clear.
	Oct. 8, '10	54	55	..	Cloudy.
	Oct. 10, '10.....	56	59	52	Clear.
	Oct. 11, '10.....	59	66	69	Clear.
	Oct. 13, '10.....	48	52	55	Clear.
	Oct. 14, '10.....	60	68	69	Fair.
	Oct. 15, '10. Finished....	61	63	..	Cloudy.
Amiesite	Nov. 7, '10 Begun.....	34	40	43	Clear.
	Nov. 9, '10	39	45	46	Cloudy.
	Nov. 10, '10	48	54	55	Clear.
	Nov. 11, '10. Finished ...	44	48	43	Cloudy.
Sicilian Asphalt.....	Oct. 19, '10. Begun.....	59	65	68	Clear.
	Oct. 21, '10. Finished....	55	57	59	Cloudy.
Tarvia Mixture	Oct. 26, '10. Begun	49	54	56	Clear.
	Oct. 27, '10.....	58	65	67	Cloudy.
	Oct. 28, '10.....	54	55	56	Clear.
	Oct. 29, '10.....	44	48	..	Clear.
	Oct. 31, '10.....	40	49	53	Clear.
	Nov. 1, '10. Finished	44	53	56	Clear.
(Flush Coat).....	Nov. 10, '10	48	54	55	Clear.
	Nov. 11, '10	44	48	43	Cloudy.
Tar and Asphalt Mixture.....	Oct. 26, '10. Begun.....	49	54	56	Clear.
	Oct. 27, '10.....	58	65	67	Cloudy.
	Oct. 31, '10.....	40	49	53	Clear.
	Nov. 1, '10	44	53	56	Clear.
	Nov. 2, '10	52	57	60	Cloudy.
	Nov. 7, '10	34	40	43	Clear.
	Nov. 9, '10	39	45	46	Overcast.
	Nov. 10, '10	48	54	55	Clear.
U. S. Tar.....	Oct. 21, '10. Begun.....	55	57	59	Cloudy.
	Oct. 22, '10.....	52	53	..	Rain.
	Oct. 24, '10.....	52	54	56	Clear.
	Oct. 25, '10.....	55	60	60	Cloudy.
	Oct. 26, '10.....	49	54	56	Clear.
	Oct. 27, '10.....	58	65	67	Cloudy.
	Oct. 28, '10.....	54	55	56	Clear.
	Oct. 29, '10.....	44	48	..	Clear.
	Oct. 31, '10. Finished....	40	49	53	Clear.

Mr. Connell.

TRAFFIC CENSUS, WHITE PLAINS ROAD. TAKEN BETWEEN THE HOURS OF 5 A. M. AND 9 P. M.

	Friday, January 6th, 1911.	Saturday, January 7th, 1911.	Sunday, January 8th, 1911.	Monday, January 9th, 1911.
Horse without vehicle	..	1	4	2
Horse vehicle, light	15	36	37	34
Horse vehicle, heavy	48	51	5	48
2-horse vehicle, light	5	8	3	6
2 horse vehicle, heavy	38	59	..	45
3-horse vehicle	1	..	..	..
4-horse vehicle	4	2	..	..
2-passenger car	2	5	11	3
4 or 5-passenger car	12	27	51	21
6- or 7-passenger car	9	12	34	9
Freight truck, omnibus, etc.	17	21	..	18
Traction engine	..	..	..	..
2-traction engine	..	..	..	..
Miscellaneous heavy traffic	..	..	..	..

Mr. Meeker.

R. A. MEEKER, ESQ.*—A great many of the so-called failures of bituminous binders are due to faulty application of the material. The speaker has seen asphalt and oils sprinkled on roads which were full of holes and hollows from 1 to 4 in. deep. When the bituminous surface had been covered with ¾-in. stone, it looked very nice, but after three weeks or a month the road would go to pieces and the bituminous material would be condemned. The speaker believes that, unless the road base is laid and thoroughly consolidated, the surface will go to pieces, no matter what kind of bituminous material or what method of construction is used.

Almost every bituminous binder that has been put on the market has been used in New Jersey. The mixing method has given very good results, but where it has failed, the foundation has always been found to be too open, and not thoroughly consolidated. The mixing method has failed in some cases because of the use of too much bitumen, as it produced a wavy surface. On the other hand, sometimes too little bitumen is used. There was one road, built by the mixing method, which broke up in several places, and it was found, by making an analysis, that only 4 or 4½% of bitumen had been used. In another road, which was inclined to be a little wavy, 8% of bitumen was found.

From the foregoing the speaker thinks that about 6 or 7% is the proper amount.

Mr. Smith.

F. P. SMITH, M. AM. SOC. C. E.—It seems strange that much of the knowledge of bituminous mixtures and construction gained during the past thirty years in laying sheet-asphalt pavements in cities is being disregarded in the construction of highways of the so-called bituminous macadam type. Surely this can only be accounted for on the ground

* State Road Supervisor of New Jersey.

that the engineers in charge of highway construction are not well acquainted with the developments of the asphalt paving industry, or that they have been misled concerning them. While all are agreed that road experience has proved that the surface of a macadam road will not withstand modern traffic, it does not seem to be fully recognized that, under present conditions, foundations and sub-drainage are even more important than in Macadam's time, when he emphasized the three cardinal principles of road making as: first, drainage; second, drainage; and, third, drainage. Mr. Smith.

Experience gained in paving city streets has proved:

1.—That from 1½ to 2 in. of bituminous wearing surface is sufficient to withstand the heaviest traffic.

2.—That when the bituminous wearing surface is composed chiefly of coarse stone, this coarse stone breaks up under heavy traffic, and the pavements as a whole soon disintegrate. The heavier the traffic, therefore, the finer, smoother and more slippery must the bituminous surface be, in order to withstand it.

3.—That uniform methods of mixing are so important that, even in the binder course, the old type of continuous binder mixer has been discarded for the modern batch mixer. Rich spots in a mixture mean soft spots in a pavement, and these soon push out under traffic and form waves and holes. Lean spots mean insufficient binding material and rapid disintegration. As soon as the surface becomes uneven, holes are rapidly produced by the pounding of traffic.

4.—That the bituminous wearing surface requires a solid and rigid foundation to support it, as it does not possess sufficient strength to bridge over depressions, and with an unstable foundation it quickly disintegrates. Even the experiment of using old granite paving blocks laid on their sides and embedded in sand as a foundation, has proven a failure in many cases; and this is a much better foundation than is being used on many highways.

5.—That a bituminous concrete foundation is more expensive than a Portland cement concrete foundation, must be of greater thickness, and, except under special conditions where excessive vibration must be provided for, is not as good as Portland cement concrete.

6.—That certain bituminous compounds are suitable for use and that others are not. Methods for the examination of bitumens have been fairly well standardized, and by these methods their availability for paving purposes can be judged with considerable accuracy. It would seem unwise, therefore, to discard these methods and the data accumulated through their use for others of doubtful value.

7.—That the mineral aggregate and method of handling is just as important as the character of the bitumen used, and that different bituminous materials frequently require entirely different methods of handling to produce successful results. Their general value, therefore,

Mr. Smith. cannot be determined successfully by parallel service tests in which all the methods of laying are exactly alike.

In spite of all this, most of these lessons are being disregarded under the plea that proper construction would be too expensive. Not only are totally inadequate foundations being used, but sufficient attention is not being paid to drainage. A macadam road with improper drainage will become a quagmire in early spring under heavy traffic, but it will reconsolidate as the subsoil dries out. Part of this drying, and under certain conditions a very large part of it, takes place by evaporation through the surface of the road. When an impervious bituminous wearing surface is laid on a road, no drying out of this nature is possible, and the bituminous surface when once broken up will not reconsolidate satisfactorily under traffic. Proper sub-drainage, therefore, is much more important with present methods of bituminous highway construction than with macadam roads.

In many cases there is an apparent attempt to combine foundation and wearing surface in what is practically one layer of imperfectly-coated large stone. As a foundation course, this layer is not thick enough; as a wearing surface, it is too thick, and the stone used in its construction is not of the proper size for either wearing surface or foundation. A bituminous concrete foundation should be composed of run-of-the-crusher stone from about 2 in. down, with sufficient fine material to fill a large proportion of the voids, and should never be less than 5 in. in thickness when the traffic is of sufficient importance to lay a bituminous macadam road. It is impossible to coat such a mixture of stone satisfactorily by the penetration method, as the bituminous cement will not penetrate the mass. Separate layers of stones of different sizes are a very imperfect substitute for this, and if poured separately are likely to produce a road which is too rich in bitumen, will bleed badly, and push out of shape. No attempt to produce a substitute for a graded mixture by the use of several separate layers of stone of different sizes coated by pouring with hot bituminous cement can ever produce a thoroughly satisfactory result.

Where very heavy traffic is to be carried by the road, the bituminous wearing surface must of necessity be composed of fine particles. Such a surface will be more slippery than one composed of coarse particles, but this may be improved by using a relatively thin layer of fine-grained particles bound together with a soft cement and applied over a properly constructed, mechanically mixed layer, composed of graded particles, and similar to the close binder course in use with most modern asphalt pavements. Such a layer will need frequent renewal, but this can be applied easily and cheaply as a squeegee coat, and this method is used successfully on certain types of pavements in many large cities. Where this squeegee coat of chips and bituminous cement is placed directly on a coarse layer of poured stone serving as a

foundation course, the top layer soon wears off, and the exposed, very coarse, and imperfectly keyed stone quickly breaks up under heavy, iron-tired traffic. The hole, when it is repaired, is usually not large enough to permit coarse stone to be used. As a result, stone chips and more bituminous cement are put on in too thick a layer, and, under traffic, they soon roll up into waves or humps. Mr. Smith.

In the speaker's judgment, the foundation course should be separate and distinct from the wearing surface, and of such a character that the wearing surface when worn out can be removed from it and replaced without disturbing it, and at minimum cost. Such a method of construction will prove the cheapest in the long run. Portland cement concrete is the best foundation of which we have any knowledge at present, and from 3 to 5 in. of it would be ample for highway work. When the subsoil is firm and well drained, the thickness of the foundation can be reduced to the minimum, but increased in those parts where the subsoil is unsatisfactory and thorough drainage is very expensive. The concrete should then be covered with a machine-mixed bituminous wearing surface of a character suited to the traffic.

Work of this character requires trained organizations, skilled workmanship, and proper plants, but so does all important work of any magnitude, and surely highway work is important enough to be done right if at all. Hauling costs can be largely diminished by the use of motor trucks, and, with machine-mixing, construction work will be much less dependent on weather conditions than it is at present.

In the long run, such a road would be less expensive than if laid by present methods. Its wearing surface would be uniformly and properly mixed, and there would be no soft spots, no bleeding, and no rolling of the surface. Even its first cost would compare favorably with roads constructed by the penetration method. Many of the New York State highways cost approximately $10 000 per mile of 16-ft. roadway, or more than $1 per sq. yd. In many cities in the United States sheet-asphalt pavements, including grading, 6 in. of Portland cement concrete, 1½ in. of binder, and 1½ in. of wearing surface, have been laid for $1.25 per sq. yd., and the form of construction proposed by the speaker, under similar conditions, should cost less than this.

Clifford Richardson, M. Am. Soc. C. E.—In regard to the merits of the mixing and penetration processes, the subject is one on which there are scarcely any accurate data to-day, and it seems to the speaker that the matter is purely in an experimental stage. Probably at the end of five years there will be sufficient data to discuss the subject rationally, but to-day the most that can be done is to draw inferences and make comparisons. Mr. Richardson.

Undoubtedly the mixing process is the more desirable, from the point of view of uniformity, and a road built in that way may be more satisfactory than one built by the penetration process at the end of the

Mr. Richardson. first three years, but in treating the mineral aggregate with the bitumen by the mixing process, only enough can be used to cover completely all the particles of rock and the finer material present. On the other hand, it is reasonable to infer that, in the penetration process, in which a larger quantity of bitumen may be used, a reservoir of bitumen may be had which can be called on to enrich the pavement during the later period of its life. This conclusion, however, is not based on any data in the possession of the speaker. The application of a flush coat each year to a road built by the mixing process would undoubtedly give the same result.

Mr. Howard. J. W. Howard, Esq.—Michigan Boulevard, in Chicago, was surfaced with graded mineral aggregate mixed with bituminous cement. The speaker observed at the end of about a year that the surface showed very little signs of bituminous material, and it was found necessary to put on a flush coat at the beginning of the second season. That flush coat acts as a sort of water-proof covering, of a rubbery nature, and prevents the surface from becoming cracked and opening voids in the stone below. This roadway is being kept up splendidly by the application of slight flush coats at intervals, although it is subjected to heavy and constant traffic.

The gas-works of England furnish vast quantities of tar, and the experiments of the English with tar products on and in roads have been extensive and valuable, but the speaker thinks that the United States has surpassed them by using asphaltic materials, which are less susceptible to oxidation than tar products.

Engineers should be warned against alleged tests which are often inserted in specifications, as they may be covert descriptions of some special brand.

Specifications should describe the essential physical and chemical qualities of bituminous or asphaltic cements, etc., without reference to brand or geographical origin.

Mr. Sargent. Paul D. Sargent, Esq.*—In 1909 six pieces of bituminous macadam, aggregating 20 700 sq. yd., were constructed under the speaker's direction. In 1910 eight pieces of the same kind of road, aggregating 19 280 sq. yd., were built. All this work was done by the hand-mixing method, the general specifications being as follows:

The roadbed was prepared as for the construction of a regular macadam road. The first course was then spread to a depth of 7 in., loose—when thoroughly rolled, this made a depth of about 5 in. The No. 2 stone, after being thoroughly coated with the bituminous material, was spread on the No. 1 course to a depth of about 3 in. After spreading and, before rolling, this course was allowed to stand, sometimes a few hours, sometimes a whole day. The rolling consolidated the course until it had a depth of about 2 or 2¼ in. This course

*State Highway Commissioner of Maine.

was then painted with hot coal-tar or hot asphalt, and immediately covered with clean stone screenings to a depth of ¼ or ½ in. The screenings were lightly rolled, and, after standing a few days, the road was opened to traffic. Mr. Sargent.

The stone used on this work was either hard limestone, coarse-grained granite, or hard trap rock. Invariably, better results were obtained with the limestone or the granite than with the trap rock. Roads built with the latter have raveled more or less, and, in some cases, the raveling began 2 or 3 weeks after their completion. In other cases, where trap rock has been used, small hair cracks have been observed on the surface after the road has been open for 4 or 5 weeks.

On most of the work, the bituminous binder was composed of a mixture of equal parts of refined coal-tar and oil asphalt. These materials have been heated separately, the tar to a temperature of about 200° Fahr., and the asphalt to 350° or 400° Fahr. The mixing has been done by hand, on wooden platforms about 12 ft. square. About ½ cu. yd. of stone is spread on the board to a depth of 6 or 7 in. The hot tar is first thrown over the stone, and then the hot asphalt. The stone is then turned about four times, and if more tar or asphalt are needed to coat the stone thoroughly, they are applied. An attempt has been made, however, to use about 1¼ gal. of bituminous material to each square yard of surface, for coating the No. 2 course. In practice, however, the quantity has generally been greater than this. For the paint coat, about ½ gal. per sq. yd. is used.

The average cost of the work in 1909, including all grading and drainage in connection with construction, was $1.10 per sq. yd. In 1910 the cost for the same work was $1.33 per sq. yd.

On some of the trap-rock sections it has been necessary to apply a flush, or paint, coat to prevent disintegration, and Tarite-Asphalt, containing about 10% of asphalt, has generally been used. Where this treatment has been given, it has stopped the disintegration. In one case it was necessary to apply this flush coat about 8 weeks after the road had been opened.

Thus far, there has been no maintenance charge of any kind, except for cleaning, on the sections built with local stone. On a road constructed of local stone and opened in September, 1909, it was noticed early in the following June that the surface coat of bitumen was pretty well worn off, and a mosaic surface was presented. The speaker watched this section quite closely, thinking it might be necessary to repaint it to prevent its disintegration. By the middle of July, however, the asphalt from the bituminous mixture having come to the surface, this section was completely sealed once more, and looked better than ever.

One difficulty in construction was experienced in attempting to use

Mr. Sargent. warm stone. This was tried a few times, particularly for the purpose of drying the stone, and in no case was the result successful. This occurred with local stone and with trap rock. In all cases where heated stone was used it was necessary to remove the section laid with it and substitute new work.

While taking a traffic census on one of these bituminous roads, a load of granite drawn by sixteen horses passed over the road, apparently without injuring it. This was in December, 1909, and the temperature was about 32° Fahr. Similar excessively heavy loads have passed over this same section of road from time to time, without causing any injury to the surface.

Mr. Blanchard. Arthur H. Blanchard, M. Am. Soc. C. E. (by letter).—The several discussions presented relative to the use of bituminous materials by mixing methods emphasize certain characteristics of pavements built by these methods and many essential elements of efficient construction.

As pointed out by Messrs. Connell, Eppele, Pillsbury, and Smith, mechanical mixing will be the ultimate economical method, and, in fact, is to-day the only practicable method of mixing broken stone with certain types of bituminous materials. The writer is fully in accord with Mr. Smith in the statement that the batch mixer is preferable to the continuous mixer from the standpoint of securing uniformity in the quality of the mix. Attention is advisedly called by Mr. Sargent to the futility of heating stone on flat plates, the pavements built with an aggregate thus heated proving unsatisfactory.

Messrs. Eppele and Smith strongly advocate bituminous pavements constructed by mixing methods in which a finely-graded aggregate is used. Mr. Pillsbury is very definite in expressing preference for bituminous pavements built by mixing methods for those cases where the traffic consists of many heavy horse-drawn vehicles.

Failures due to the use of trap rock of certain sizes in the mixing method are cited by Mr. Sargent. The writer has in mind pavements constructed with uniform large-size crusher-run trap rock which have failed in a similar manner elsewhere.

The self-evident effects of using too small a quantity of bituminous material, resulting in the disintegration of the road in spots, and the excessive use of bituminous material, causing a wavy surface and the bleeding of the road, are well brought out by Messrs. Meeker and Smith. In connection with the figures quoted by Mr. Meeker, the writer might add that, in a bituminous pavement recently inspected, the portion constructed with 6½% of asphaltic binder was in excellent condition, while the sections constructed with from 7 to 8% had bled.

Messrs. Meeker and Smith also call attention to the importance of adequate foundation and drainage, if satisfactory bituminous pave-

ments are to be expected. It is extremely unfortunate that more attention is not paid to the drainage and foundations of roads and streets, as many failures coming under the writer's notice have been due to one of these causes. Mr. Blanchard.

The writer thoroughly agrees with Messrs. Howard, Richardson, and Smith as to the value of a periodical application of a flush or seal coat. As presented by Mr. Smith, and as emphasized by the writer in the opening discussion, the function of a periodical seal coat has not been fully appreciated by American engineers.

Mr. Connell has rendered a valuable service to American engineers in presenting detailed cost data and complete information relative to the construction of experimental sections of various kinds of bituminous pavements built under conditions existing in large municipalities.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1195

THE PITTSBURG AND LAKE ERIE RAILROAD CANTILEVER BRIDGE OVER THE OHIO RIVER AT BEAVER, PA.*

By Albert R. Raymer, M. Am. Soc. C. E.

With Discussion by Messrs. C. W. Hudson, Henry S. Prichard, Albert B. Hager, F. W. Skinner, A. W. Carpenter, and Albert Lucius.

History and General Information.

As the business of a railroad increases, it becomes necessary in many cases to change its facilities, and frequently a change in one department necessitates many and serious changes in others. The Pittsburg and Lake Erie—the road of the New York Central Lines into Pittsburg—is a striking example, for its management has spent vast sums of money to keep it abreast or a little ahead of the requirements of increasing traffic. Its bridge over the Ohio River at Beaver, Pa., built in 1878, had to be renewed by 1890, which was done on the single-track masonry by building a bridge which at that time was thought to have strength sufficient for many years. It was built for loadings equal to Cooper's E-30, with low unit stresses.

About 1897 the locomotives were increased in weight enough to allow the train tonnage to be increased from 1 600 to 2 500, and about 1902 the locomotives were again increased, with a corresponding train tonnage increase to 3 500, and since that date it has been increased to

* Presented at the meeting of March 1st, 1911.

THE PITTSBURG AND LAKE ERIE RAILROAD CANTILEVER BRIDGE OVER THE OHIO RIVER AT BEAVER, PA.

more than 4 200. With this increase in tonnage, the tracks have been changed: from a single-track to a double-track approach to the bridge, with a single track on the bridge; then to a single-track gauntlet; and then to a four-track construction to points near the bridge. These conditions made it desirable to consider the reconstruction of the bridge, for strength as well as for increased capacity. The existing masonry piers were not large enough to support a double-track bridge, and, as they were constructed on timber grillage resting on gravel, it would not have been an easy matter to have enlarged the piers under traffic; and, in addition, the River Interests desired the removal of one of the channel piers which was considerably in the way when large coal fleets were being handled at that point. After a careful study of the situation, a new bridge on a new location was recommended and approved in 1906. The new location is about 300 ft. up stream from the old bridge (Plate I), and the approaches are on 3° curves, instead of 6°, as in the old approaches.

The Ohio River Bridge Laws require 90 ft. in the clear above low water, in the district in which this bridge is located. The bridge is on a summit in the grades, with a maximum grade approach of 0.30% on one side for about 2 miles.

The Government work of improving the river above and below the bridge site is planned for a channel through the bear-trap dams 700 ft. wide. This width is provided in the clear at the new bridge at right angles to the channel. After allowing for the skew and the thickness of the channel piers, the channel span was defined at 769 ft. from center to center of bearings. The channel span was located by the United States Government Engineers—after several meetings had been held to determine the wishes of the River Interests—with one pier just back of a timber crib on the south or left shore of the river. The length of the channel span, and the stipulation by the Government that no false-work should be placed in the channel in the river, practically determined the design to be a cantilever, and the location of the channel span forced one arm entirely over land, making it necessary, much to the regret of the Railroad Engineers, to use an approach span at the north or Beaver end. It was shown by the Railroad Engineers that floats, liberated at the coal-boat stage of water in the center of the channel a mile above the proposed structure would strike the channel pier, and yet the rivermen preferred the old channel, close to shore, defined

largely by the location of the offending pier in the old structure which was to be removed.

The spans and the general outline of the proposed bridge were determined by the Railroad Company's Chief Engineer, J. A. Atwood, M. Am. Soc. C. E., and the plans and general drawings were developed by Albert Lucius, M. Am. Soc. C. E., Consulting Bridge Engineer for the Railroad Company.

General Features of Design.

The general design of the new bridge called for two gauntlets of two tracks each to accommodate the four tracks at each end of the bridge, the northward passenger and the northward freight forming one gauntlet, and the two southward tracks making the other. The tracks in each gauntlet have their adjacent rails 1 ft. apart from center to center, both freight tracks being toward the center of the bridge and 13 ft. apart, the passenger tracks, therefore, are 15 ft. apart. The side clearances are 7½ ft. from the centers of the passenger tracks to the trusses, and 7 ft. to the skid girders; the overhead clearance is 21 ft. 6 in. from the top of rail.

The general loading assumed is Cooper's E-60 for the floor, and the same with a 10% reduction for the trusses.

As the E-60 loading, with full allowances for impact, wind, etc., is considered sufficient to provide for all estimated increases in train loads, rather high unit stresses were used.

River clearances and grade conditions made it desirable to have a shallow floor. The depth finally determined was 6 ft. from the clearance line to the top of rail. This depth practically determined the panel lengths for good stringer and floor-beam construction. The anchor arms were divided into ten panels of 32 ft. each; the cantilever arms into four panels of 32 ft. and four of 28 ft. 6 in. each; and the suspended span into ten panels of 28 ft. 6 in. each. This made the length of the anchor arms 320 ft., the cantilever arms 252 ft., and the suspended span 285 ft. The approach span is 370 ft. long, and is divided into twelve panels, each 30 ft. 10 in. long. The trusses are 34½ ft. apart from center to center, and 30 ft. apart in the clear. The floor is made up of nine lines of stringers, four under each gauntlet track and one half way between. A special feature of the floor is its strength, which is designed to take care of derailments under all probable conditions. Skid girders—the design of the Chief Engineer

FIG. 1.—BOTTOM CHORD, L_6 L_8, BRIDGE AT BEAVER, PA.

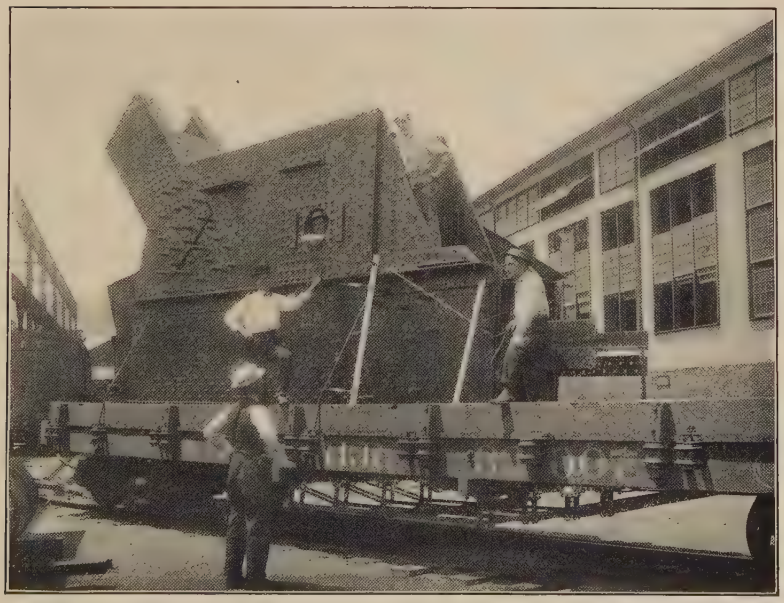


FIG. 2.—PART OF BOTTOM CHORD, L_{16} PANEL POINT.

of the Railroad Company—have been used to prevent derailed equipment from striking the trusses, the object being to cause the car bodies to "skid" on the floor or against the skid girders until the momentum will have been destroyed. These girders are double on each truss, and are placed at the proper height to give the best protection to the truss members.

The magnitude of the bridge, and the heavy loading, developed enormous stresses, which made it necessary to give much more than the usual attention to many details of the design, which will be considered at length at the proper place. Some of these features were developed in designing the anchorage, in transferring the wind stresses to the anchor masonry, and in the construction of the main-post bearings on the masonry. At the anchor points the anchor eye-bars are maintained under maximum stress at all times. The change in the position of the L_0 point over the anchorage, due to expansion and contraction, is prevented from causing bending moments in the eye-bars; the change in stress at the L_0 point from positive to negative does not cause any pounding, and the wind stress does not cause any stress at the anchor masonry, sidewise or otherwise, in the anchor eye-bars, when the abutment reaction is zero. These points have been protected by very ingenious details.

The main-post supports on the channel piers are unique in their design, in distributing the loads of 6 000 tons at each bearing definitely on the masonry under all conditions, and at the same time they prevent the development of secondary stresses of any serious magnitude in the lower chords at these points, due to changes in the live loads in either magnitude or position.

The timber deck design has been made to serve, not only under normal conditions, but also to offer a large degree of safety under derailed equipment; the ties are well supported under the probable locations of derailed wheels, and they are prevented from bunching under such conditions. The timber in the deck has been treated chemically.

The anchor arm being comparatively long, the reaction at its end from dead load is very small, causing an uplift of only 33 000 lb. The uplift per truss at the L_0 point, caused by the live load and impact, amounts to 2 299 000 lb., and the load at the same point is 2 071 000 lb. This causes a reversal of stress in the web members up

to and including L_2, U_4, in the top chords up to U_6, and in the bottom chords up to L_4. The connections, L_0, L_2, U_2, and U_4, therefore, are riveted throughout. In all other cases the post and hanger connections to the bottom chords are riveted, while the main diagonals are connected to the bottom chords by pins back of the floor-beam connections, shown on the typical details of the panel points, L_{14} and L_{16}, Plate II. This was done in order to get an efficient connection of the floor-beams to the trusses, which otherwise, on account of the small available depth of 6 ft. from the top of rail to the clearance line, would have been very difficult, if at all possible. The members of the secondary system are also riveted to the bottom chords. In order to make the trusses of the anchor arm self-supporting during erection, and before the cantilever arm was erected, U_6-U_8, was also designed as a stiff member. It was constructed as a riveted four-web chord with pin connections at both ends, all other top chords being two-web chords. This provision was a source of great comfort when high water and driftwood threatened the falsework on the Beaver side after the anchor arm had been swung, and before the cantilever was built, it having been just started.

The riveted connections in tension and in compression are fully spliced; no allowance was made for butt joints, with the exception of the member, L_{10}-U_{10}, where, in addition to the straight butt joint, only half of the stress was taken care of by splice material and rivets. This member being vertical, the individual pieces forming it naturally had to come to a full bearing by their own weight.

The bottom chords have an **H**-section, made up of twelve angles and two plates throughout. The heaviest bottom chord has 644½ sq. in., and is made up of three 60 by $\frac{7}{8}$-in. web plates on each side, outer angles 8 by 8 by $\frac{3}{4}$-in., and inner angles 8 by 8 by $\frac{7}{8}$-in., at top and bottom of the chord, and two lines of 8 by 8 by $\frac{7}{8}$-in. angles on the inside of the chord.

The bottom chords have three sets of 6½ by $\frac{9}{16}$-in. double lacing, at the top and bottom, and a double lacing of 7 by 3½ by $\frac{7}{16}$-in. angles between the angles on the center line of the chord. Wherever the lacing connects the inner angles of the top and the bottom flanges, tie-plates are used to connect with the outer angles. Vertical diaphragms, at from 5 to 6-ft. centers, are used in these chords throughout. The intermediate diaphragms are made in two separate parts in a vertical plane to clear the middle lacing. At the panel points the diaphragms

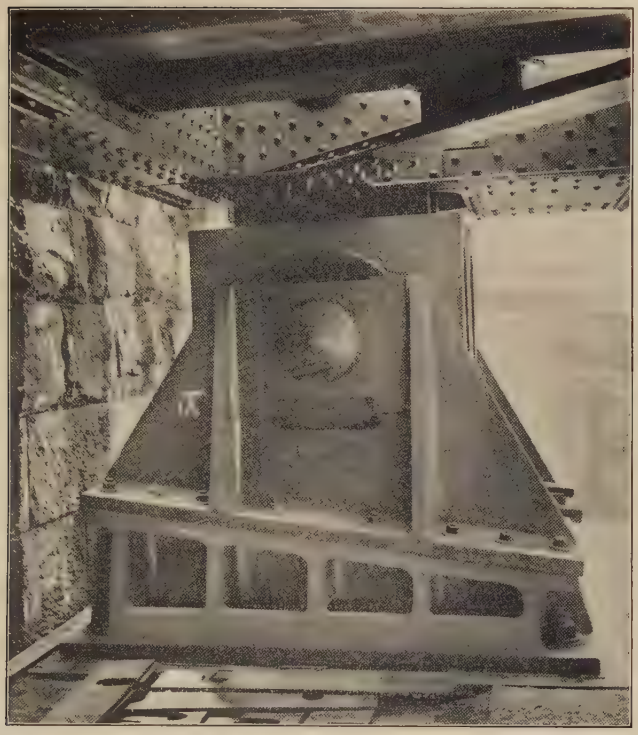


FIG. 1.—ANCHORAGE DETAILS, BRIDGE AT BEAVER, PA.

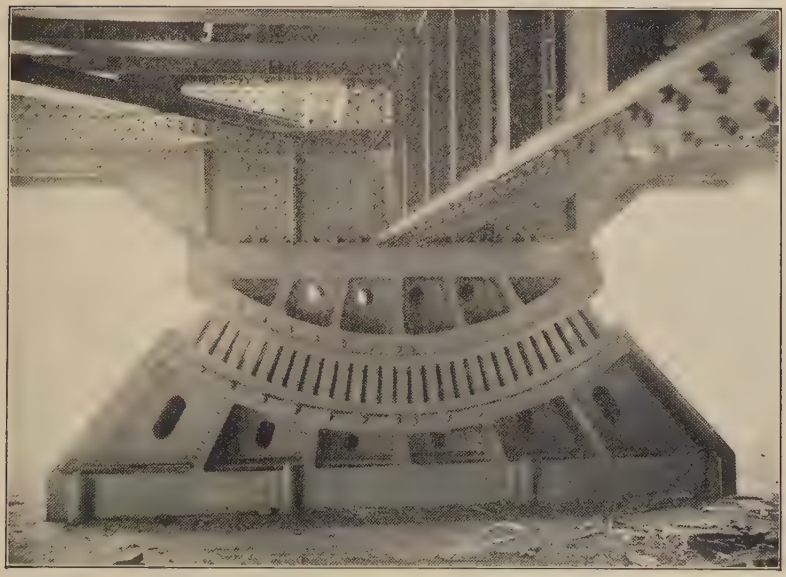


FIG. 2.—CAST-STEEL BEARINGS FOR TOWER POSTS.

are framed around the chord angles at the center of the chord, and riveted together on the tie-plate at the panel point. The chords are 3 ft. 6½ in. from back to back of the outer flange angles throughout.

The splices of the lower chord were detailed so as to permit of milling together all joints, for the purpose of maintaining exact lengths by the introduction of interior follower or splice-plates, which were not shop-riveted to either section of the splice. These facilitated the proper assembling of all sections in the shop and in the field, and yet distributed the splice values of section and of shear over more than one point, and put rivets in double shear. The connections of the web and the chord members were through large plates treated in a similar way. All connection plates were proportioned, not only for section and rivet shear in axial direction, but for the necessary section along shear lines between the several members assembled to a main joint, and also for the symmetrical absorption of stress in relation to their axes and to the resultant, through rivet values. The secondary bending and shearing stresses developing in large attachment plates, due to deviations of centers of gravity, were considered and guarded against by additional reinforcements.

The general design of the bridge provides for trusses riveted for full section of all joints and splices of chords, posts, and individual members, excepting as noted for the main-tower posts. Connections of chord and web members exposed to reversal of stresses are riveted joints, and so are all sub-panel connections and the end posts of the cantilever arms. Connections of chord and web members having stress in one direction only are pin-connected. In all the heavy members the lacing is on the inside. Single-angle lacing is also turned in and connected with the inside of the flanges.

All heavy members have vertical diaphragms every 5 or 6 ft. The diaphragms in all cases were planed to the exact width, and the sides of the members were assembled to them in their true position, the lacing being put on afterward. On account of this, in no case was any trouble caused by a twisting tendency while the members were being assembled, handled, or erected.

The trusses of the anchor arms are 64 ft. high at the hips (the U_2 points), 145 ft. 2½ in. over the main piers (the L_{10}-U_{10} post) and 62 ft. 0 in. at the ends of the cantilever arms (the U_{18} point) from center to center of chords. The suspended span is 57 ft. 0 in. deep,

and the approach span is 61 ft. 8 in. at the hips and 75 ft. 0 in. at the center, all from center to center of chords. In the cantilever bridge the trusses are 34 ft. 6 in. from center to center, and in the approach span they are 34 ft. 0 in.

Drawings and Proposals.

The Railroad Company furnished the specifications, the design, the stress sheets, and detail sheets developing each joint sufficiently to determine the construction, the shape, and the style of each member and all connections, for the purpose of estimating the weight of the material and the cost of fabrication and erection of the bridge.

Contractors were invited to submit alternate plans and proposals. None, however, was offered. The contract was awarded on the Railroad Company's design and specifications, which, in most of its details, was in the usual form. Profiting, however, by the experience of others in a recent disaster, it contained the following clauses which were actively operative.

"It is expressly understood and agreed between the parties hereto, that the Contractor shall be wholly responsible for the safe and satisfactory erection of the whole superstructure, and for this purpose the Contractor is required to, and does hereby expressly assume the responsibility for the correctness of the design and for the material being of good quality and properly fabricated, notwithstanding any previous inspection or acceptance of the material by the railroad.

"The Contractor shall execute and deliver to the Railroad, before any material is erected, its two bonds, satisfactory in form to the Railroad, and with such Surety Company as surety thereon as shall be approved by the Railroad. One of said bonds shall be conditioned for the faithful performance of this contract in so far as it applies to the fabrication, erection, and finishing of the cantilever span, and the other of said bonds shall be conditioned for the faithful performance of this contract in so far as it applies to the fabrication, erection, and finishing of the approach span. Each of the said bonds shall be in an amount equal to the total estimated payments which will be required by the Contractor from the Railroad before the completion of the respective spans. The Railroad will reimburse the Contractor in the amount of one-half of the premiums paid by it on said bonds."

The specifications, in most particulars, were not unusual, excepting in the eye-bar requirements, the substance of which is given elsewhere.

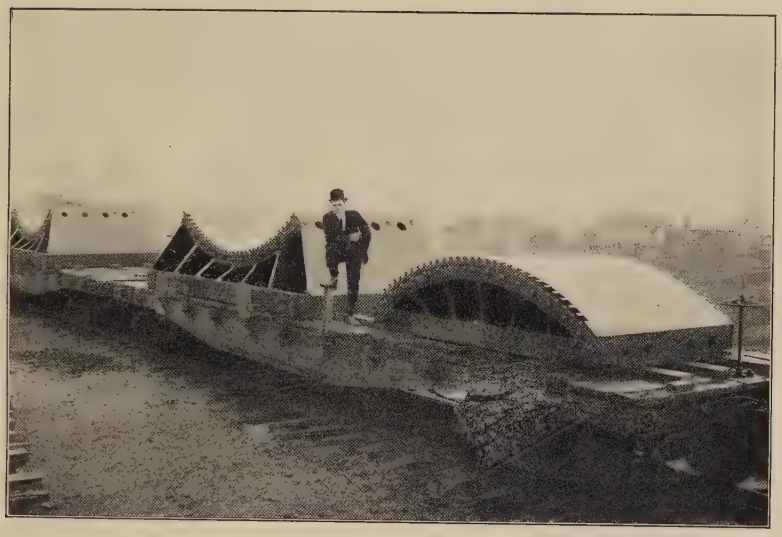


FIG. 1.—UPPER AND LOWER CASTINGS SUPPORTING TOWER POSTS.

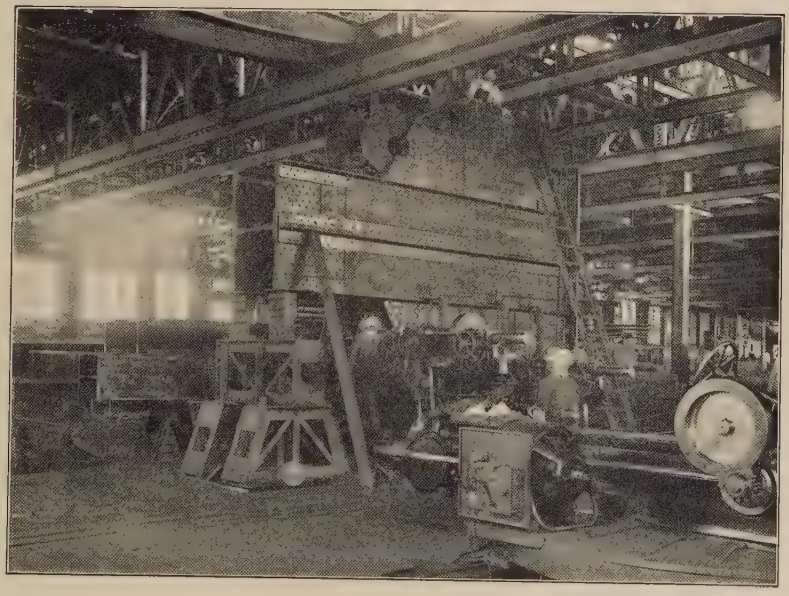


FIG. 2.—L_{10} "STUB," IN PROCESS OF MANUFACTURE.

Loading.

The floor system was proportioned for carrying two trains, one on each track, each weighing 6 000 lb. per lin. ft. and each preceded by two locomotives weighing 426 000 lb. each.

Two trains were assumed on each track wherever split loading produced greater results.

The trusses were designed for loads 10% less than these. The dead load averaged about 9 tons per lin. ft., of which ½ ton was for the timber deck and the rails. The dead-load stresses were computed from weights figured from the stress sheets. This method was repeated until the sections, computed on the basis of the actual dead load, figured from the shop drawings, came within 2% of the sections used.

The wind loads were proportioned by the areas of the exposed surfaces as finally developed, and were equivalent to about 1 100 lb. per lin. ft. of the lower chord for the anchor and the cantilever arms, about 1 000 lb. per lin. ft. for the lower chord of the suspended spans, and about 600 lb. per lin. ft. for the top chords. The wind loads were computed from the assumption of 300 lb. per lin. ft. on a train, and 30 lb. per sq. ft. on the exposed surfaces of the two trusses and the floor. Of this wind pressure, 10 lb. per sq. ft. was treated as being uniformly distributed over the whole structure, while the remaining 20 lb. was used as a moving load, applied wherever it produced the maximum effect on the members.

Unit Stresses.

The axial tension of the net section was assumed at 16 000 lb. per sq. in. The axial compression per square inch of gross section was assumed to be

$$\frac{16\,000}{1 + \dfrac{L^2}{12\,000\,R^2}},$$

but not to exceed 14 000 lb.

The impact allowed was:

$$I = L\,\frac{L}{D + L}$$

where I = Impact,
D = Dead load,
L = Live load,

excepting that impact in hangers was assumed to be 100% of L.

For wind stress, 20 000 lb. in tension was used, and a corresponding amount in compression, but not to exceed 18 000 lb. per sq. in.

For dead load, live load, impact, and wind load the units were 22 000 and 20 000 lb., respectively.

For erection stresses, 18 000 and 16 000 lb. for tension and compression, respectively, were assumed.

The pressure on concrete containing steel grillage was limited to 400 lb. per sq. in.

The bearing on the 12-in. rollers forming the bearings under the towers was limited to 6 000 lb. per lin. in.

In members subject to reversal of stresses, the resultant tensile stress and the resultant compression stress were determined, and each was increased by 80% of the smaller; the member was then designed so as to be capable of resisting either of the increased stresses. Shear in shop-driven rivets was taken at 11 000 lb. per sq. in. For hand-driven field rivets, the number was increased 25% over the number needed when shop-driven, and for hammer-driven rivets the number was increased 10 per cent.

Rivet bearing was figured at 22 000 lb. per sq. in., and bending on pins was taken at 24 000 lb., with corresponding reductions for field-driven rivets.

Anchorage and L_0 Details.

Under the specifications, the contractor was required to work out and furnish shop plans and details of all parts of the bridge structure. These details were extremely well designed, and the plans were very elaborate; in fact, one of the chief features of the construction of this work was the very unusual amount of attention given to the design of the details.

The anchorage stresses varied from 2 071 000 lb. compression per truss to 2 299 000 lb. tension. The L_0 pin point of the truss is 5 ft. 1¾ in. above the pin point at the upper ends of the eight 14 by 1½-in. eye-bars forming the anchorage for each truss. These pin points are connected by a rocker. The total motion of the L_0 point, from temperature and live-load changes, is 5 in., or 2½ in. on each side of the anchorage eye-bar pin center. This inclination (2½ in.) of the rocker out of the line of the anchor eye-bars produces, under extreme conditions, a horizontal thrust of about 100 000 lb., which, if not taken care of, would cause bending of the anchor eye-bars where

FIG. 1.—PUTTING IN PLACE THE PIN SUPPORTING DIAGONAL MEMBER.

FIG. 2.—PUTTING IN PLACE THE PIN SUPPORTING DIAGONAL MEMBER.

they leave the masonry at the bridge-seat level. To overcome this condition, a cast-steel jaw is bolted securely to each side of the lower part of the cast-steel bearing, which is anchored securely to the masonry (Fig. 1, Plate III). These jaws engage slides, one at each end of the pin in the upper ends of the anchor eye-bars. Under this arrangement, this pin can move vertically to any position desired, and at the same time the horizontal thrust from the rocker is transmitted through the pin to the slides, which find support against the jaws, thereby transmitting the stress to the masonry and entirely relieving the eye-bars from bending.

The rockers which connect the L_0 pins with the anchor eye-bar pins are solid links, designed to take care of the reversal of stresses mentioned and for rotation about the two 14-in. pins under load as the L_0 point travels backward and forward. This is accomplished by using cast-steel bushings lined with manganese-bronze around each pin.

The anchor eye-bar pins are supported in a cast-steel block resting on an adjustable wedge on the masonry bridge seat. This construction transmits the positive load from the L_0 point to the masonry, without putting any load on the anchor eye-bars. After the completion of the bridge, a special live load was placed on the suspended span and on the cantilever arm, in excess of the working live load, which condition produced an uplift on the anchor eye-bars in excess of that produced by working conditions, and, while the bars were thus elongated, the wedges under the eye-bar pin supports were forced tight, by driving and by special set-screws, and secured in this position. This design maintains a constant maximum tension in the anchor eye-bars. After the bars were placed under this stress, the openings in the concrete masonry around the bars were filled with soft concrete and sealed.

The anchorage in the masonry at the lower ends of the anchor eye-bars is made with large inverted shoes with 14-in. pin bearings.

I-beam grillage is built in the masonry just above these shoes to engage fully the masonry load; these inverted shoes were put in place as the masonry was being built; set-screws were used in leveling and adjusting them to exact lines and levels. Vertical shafts were formed in the concrete masonry in order to allow the eye-bars to be placed after the completion of the masonry. The lower pins were

entered ready for driving, and a horizontal passageway or tunnel was left at the lower pin elevation to allow access to the pins. A vertical manhole or shaft, about half way between the two sets of eye-bars, connected this tunnel with the bridge seat more than 50 ft. above.

Several months after the completion of the masonry, check measurements were made from the bridge seat down to the lower pins for ordering the eye-bars of exact length. The bars were put in place and the pins driven without any trouble.

Details of the Tower-Post Bearings on the Channel Piers, and of the L_{10} Points.

Originally, the bearings of the towers on the main piers were designed to be solid, without any rollers, and the contract was let with the plans calling for this construction. Considerable discussion and some criticisms were developed on this point at the time the various bridge companies were bidding on the work, and, as a result, the Railroad Company furnished a revised plan, calling for radial roller bearings rotating around the intersection point of the lower chord with the tower posts—the L_{10} point—on a 9-ft. radius, with the object of reducing the secondary bending stresses from counter-flexure of the lower chords, and bending of the tower posts. This design delivers only vertical reactions centrally on the masonry piers. These bearings transmit to the masonry a loading of 12 558 000 lb., including live load, impact, and dead load. As a fixed bearing, calculations showed that secondary stress would be produced in the chord, L_9-L_{10}, ranging from $\pm$ 18 000 to $\mp$ 19 000 lb. per sq. in., and in the chord, L_{10}-L_{11}, ranging from $\pm$ 5 000 to $\mp$ 25 000 lb. per sq. in., and at the same time the masonry pressure was increased from an average of 400 to a maximum of 1 000 lb. per sq. in., which fully warranted the change in design as made, as an extra. This design reduced the secondary stresses in the chords mentioned to less than 3 000 lb. per sq. in. Each bearing consists of eighteen 24-in., 120-lb., I-beams, built flush in the concrete masonry for grillage. Sheet lead, $\frac{1}{4}$ in. thick, was used on this grillage under the cast-steel bearings. These castings (Fig. 2, Plate III), are in one piece, 18 ft. long and 8 ft. 6 in. wide, and weigh about 37 tons each. The tower post is supported on an upper casting, and between the two are 29 segmental rollers, 4 in. wide, 12 in. in diameter, and 72 in. long. The upper casting is turned to a radius of 8 ft. 6 in., and the lower to 9 ft. 6 in. Care-

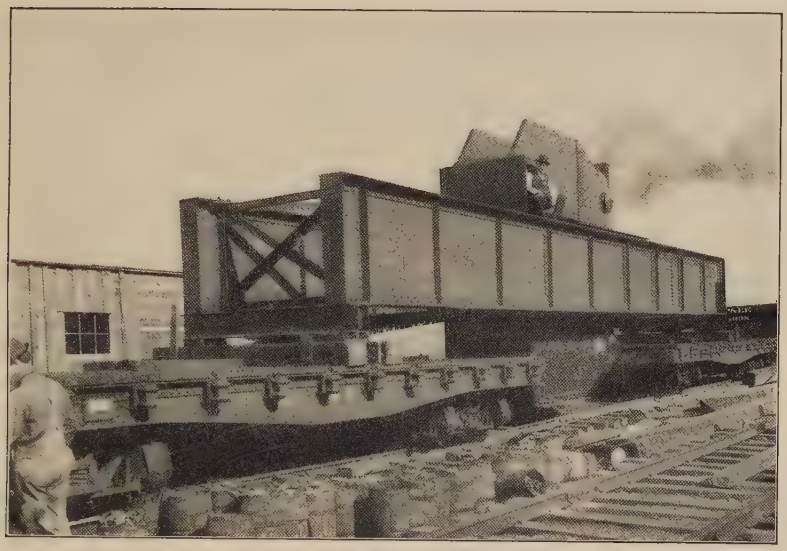


FIG. 1.—METHOD OF TRANSPORTING "STUBS."

FIG. 2.—SHOP ASSEMBLING.

ful inspection showed that these surfaces were machined correctly to within 0.002 in. A variation of 0.003 in. for low point was allowed on the rollers, but no point was allowed higher than the nominal diameter of the roller. The rollers are held in place by gear teeth at each end of each roller, and these engage in suitable racks bolted one to the upper and the other to the lower casting. By actual tests, made during erection and since, the motion of the rollers has been found to correspond closely with the estimated amount. The upper casting, Fig. 1, Plate IV, bearing directly on the rollers, weighs about 25 tons, and supports the tower post, which it fits and to which it is bolted.

The L_{10} stubs, Fig. 2, Plate IV, consisting of the L_{10} point with portions of the chord, L_9-L_{10} and L_{10}-L_{11}, and a portion of the bottom of the posts from the bearing on the castings to a splice point a short distance above the chords, were extremely difficult to make. In addition to the chords and the post being spliced solidly together, the plates, forming the L_{10} point, support by pins the bottoms of the diagonal members, U_8-L_{10}, and U_{12}-L_{10}, respectively, as shown on Plate V. These "stubs" had four planed bearing surfaces and two pin bearings, all of which had to be located exactly right. These pieces, when finished ready for shipment, were 20 ft. 2 in. long, 15 ft. 10 in. deep, and 5 ft. 3 in. wide, and weighed 55 tons each. They were handled by rail, suspended between two 50-ft. girders, between the ends of cars supporting the ends of the girders, as shown by Fig. 1, Plate VI.

The sectional areas of the lower chords are spliced through from side to side of the L_{10} stubs, and the sections of tower posts above the chords are carried through the chords to the cast-steel bearings.

In this manner the horizontal and vertical stresses have their independent sections, and the whole mass is thoroughly connected by splice-plates with full shear values on all lines on which shears can develop. For erection purposes, the inclined members were connected to this point by pins.

Specifications.

The specifications applied generally, and were substantially adhered to.

Cast Steel.—Cast steel was required to show by specimen test from 65 000 to 70 000 lb. ultimate, with the elastic limit 50% of the

ultimate; the elongation in 2 in. to be 20%; the reduction of area 45%, and a cold-bending test of 90°, to a radius equal to the diameter of the test piece. No trouble was experienced in getting such results; the elongation was generally 25 per cent.

Cast steel requires an annealing temperature of 1 700° Fahr., and large pieces must be kept soaked at that temperature for from 5 to 6 hours and then must be kept covered so as to cool slowly and uniformly in from 48 to 60 hours, in order to insure a good result. Proper annealing is considered an essential in producing the desired result. Some of the pieces were condemned on account of the test pieces showing a coarse granular fracture; these were reannealed, after which the tests were entirely satisfactory.

The composition of the steel required to produce 70 000 lb. ultimate was about as follows:

Carbon	0.27 to 0.30
Manganese	0.65 to 0.70
Phosphorus	0.035
Sulphur	0.040
Silicon	0.27 to 0.30

Compression tests on cubes showed a low yield point. The first permanent set occurred at 13 000 lb.; the material commenced to flow at 120 000 lb., and at 250 000 lb. the cubes were reduced to about one-half their original height and the sides had increased from 1 in. to 1.43 in. without showing any fractures or other signs of breaking up.

The friction bushings of links, suspenders, and sliding bearing plates were of manganese-bronze. This material was required to show a test of not less than 75 000 lb. ultimate, not less than 35 000 lb. elastic limit in tension, and 28 000 lb. in compression, and not more than 10% set under a compression of 100 000 lb. per sq. in. It was also required to stand a cold-bending test through 120° to a radius equal to the thickness of the piece tested. The elongation required was not less than 25% in 2 in., and with a reduction of not less than 25% of the area. There was no difficulty in getting metal to meet these requirements.

The tests showed 80 000 lb. ultimate, and 42 000 lb. elastic limit, with compression close to the specification requirements. The mill-finished specimens would bend 180° and then spring back to 135 degrees.

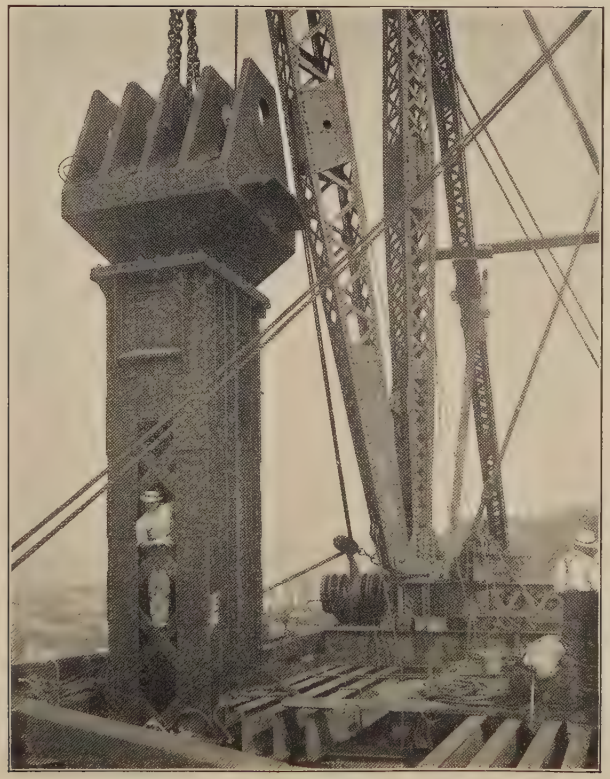


FIG. 1.—MAIN TOWER POSTS DURING ERECTION.

FIG. 2.—MAIN TOWER POSTS DURING ERECTION.

Eye-Bars.—The eye-bars were rolled at the Homestead Plant, from ingots which weighed from 9½ to 10 tons. From 35 to 40% was broken down and trimmed from the upper end, and from 10 to 15% from the lower end, of each ingot; that is, only about 50% of the ingot was used for eye-bar material. Only one of the largest bars came from one ingot, and, of the smaller bars, two came from one ingot. The slab which was used was rolled in the first heating to approximately twice the cross-section of the bar; it was then re-heated and finished to the proper size. The material was made into eye-bars at the plant of the American Bridge Company, at Ambridge, Pa., under the methods usually followed there. The bars were annealed by being heated slowly to a temperature of from 1 500 to 1 600° Fahr., and, when up to that temperature, were removed from the furnace and placed in a muffler open at each end; here they were allowed to cool until red heat was reached, then they were removed from the muffler and allowed to cool finally in the open air.

Eye-bar material was criticized only in a preliminary way by the specimen tests, which required it to be uniform and within certain limitations. The bars were accepted only as finished bars on the results of the representative full-sized tests.

Pins were drilled from end to end through their axes, and rough-turned before inspection and acceptance of the material.

The specifications also required all splices of members, and all joints and connections, to be shop-assembled in their correct positions, and while in that position all connection holes were reamed or drilled.

Details of the Main-Tower Posts, L_{10}-U_{10}, and of the U_{10} Casting.

The main-tower posts extend from the cast-steel bearing under the lower chord up to the cast-steel capping at the U_{10} point. This capping takes the reactions from the upper chord eye-bars, 16 of which, measuring 16 by 1⅞ in., are on each side of the 16-in. pin in the cast-steel cap. This casting, Plate VII, is in one piece, and weighs 37½ tons. It is provided with shelf-plates for the connection of the upper transverse strut on the axis of the pin, which forms part of the lateral bracing of the top chords. The main pin is 8 ft. 2½ in. long, and is supported on the cap by five vertical cast-steel ribs. The cap is 8 ft. 6 in. high, 7 ft. 6 in. wide, and 8 ft. 0 in. long, and is bolted to the top of the main post.

The main posts, L_{10}-U_{10}, have a cross-section of 640 sq. in. They are made up of a continuous diaphragm consisting of two 36½ by ⅞-in. web plates and four 8 by 8 by ¾-in. angles, three 66 by ⅞-in. web plates on each side, with 8 by 6 by ⅞-in. flange angles on the outside and 8 by 8 by ⅞-in. flange angles on the inside. Central with the diaphragm, on the outside of the plates, are placed a 17½ by ⅞-in. plate and two 8 by 6 by ¾-in. angles, the 8-in. leg being against the main plate, and two 6 by $\frac{9}{16}$-in. fillers are between the outstanding 6-in. legs. Double 6½ by ⅝-in. lacing bars are used on the inside of the flange angles, and are tie-plated over the outer flange angles, similar to the chord construction. These posts were made up in three sections above the L_{10} stub. The total weight of each post is 224 tons.

Shop Assembling.

The main-tower posts, L_{10}-U_{10}, including the L_{10} stub, were assembled in the shop after the butting joints had been most carefully faced. The splices were drilled after the sections were in proper alignment and the joints had been drawn up to tight contact. In drilling these and other shop-assembled splices, the members were laid down with the inside face up. About every sixth hole of the splice or connection had been punched or drilled through all the material in order to allow assembling bolts to be used. All other holes were blank, except in the upper layer of plates or angles of each side, as it was lying in the shop, which was the outside layer of the inside web and the inside layer of the outside web. All holes were punched or drilled small, and, with these holes as a template, all the other holes were drilled through them; first short drills were used in the inner webs, then long drills were inserted through these holes and guided by them; these drills then entered the holes of the inner layer of plates or angles of the outside web, and were guided by them in passing through the remainder of the material. The holes, which had been previously provided all the way through the material for bolting, were reamed after the other holes were bolted up. When shop-assembling the trusses, the members were lined with a transit to the estimated dead-load deflection after erection.

After the posts, L_{10}-U_{10}, were drilled, the bottom chords, L_0-L_{10} and L_{10}-L_{18}, with the L_{10} stub, were assembled (Fig. 2, Plate VI) and properly lined up in a position corresponding to the deflection under dead load in the structure, in which position the splices were drilled,

No. of test.	Heat number.	Slab number.	Specimen Tests: Annealed.				Chemical Analysis.				Member.	Contract number.	Remarks.
				Ult. Str.	El. Lim.	Ult. Str.	Car.	Phos.	Mn.	S.			
1	48 443	241 848	30	62 470	35 400	62 900	0.24	0.024	0.65	0.032	U_5–U_9	3 599	
2	41 006	1 784	30	61 920	36 380	57 810	0.28	0.010	0.53	0.030	U_5–U_9	3 599	
3	48 455	244 852	30	63 360	36 520	63 360	0.27	0.015	0.55	0.034	U_8–M_7	3 599	Did not break.
4	41 023	7 201	20	66 120	37 450	65 820	0.24	0.015	0.64	0.030	$L_{0\,A}$–$L_{0\,B}$	3 599	
5	48 445	242 340	30	58 740	36 140	59 800	0.24	0.030	0.65	0.031	$L_{0\,A}$–$L_{0\,D}$	3 599	
6	49 452	242 511	30	58 090	35 600	63 360	0.28	0.020	0.61	0.033	$L_{0\,A}$–$L_{0\,C}$	3 599	
7	41 013	3 817	10	57 610	35 720	57 670	0.25	0.015	0.55	0.023	U_5–U_9	3 599	
8	41 008	2 311	10	59 020	36 230	57 980	0.25	0.014	0.53	0.028	U_5–M_5	3 599	
9	50 300	168 353	10	58 160	37 040	59 660	0.24	0.015	0.59	0.028	U_5–M_5	3 599	
10	49 464	245 501	30	57 450	35 940	58 340	0.25	0.012	0.56	0.031	L_6–M_7	3 599	
11	44 038	15 330	20	60 600	37 180	60 980	0.24	0.019	0.65	0.025	U_{11}–U_{12}	3 599	
12	44 034	13 415	30	62 620	36 800	61 880	0.24	0.030	0.65	0.025	U_{11}–U_{12}	3 599	
13	44 039	15 865	10	60 870	33 380	59 290	0.27	0.011	0.61	0.032	U_{10}–U_9	3 599	
14	44 009	2 635	10	67 340	37 900	65 960	0.25	0.018	0.56	0.030	U_{10}–U_{11}	3 599	
15	41 014	4 223	10	58 960	37 010	56 860	0.25	0.014	0.56	0.030	U_{10}–U_9	3 599	
16	49 462	244 916	10	61 940	36 690	60 180	0.28	0.012	0.60	0.031	U_{10}–U_{11}	3 599	
17	40 099	34 655	20	60 900	34 780	60 060	0.26	0.010	0.53	0.027	U_{13}–U_{14}	3 601	
18	44 073	26 584	20	59 160	35 870	64 140	0.27	0.018	0.62	0.028	U_{13}–U_{14}	3 601	
19	44 064	23 608	10	60 040	35 640	56 840	0.26	0.012	0.63	0.030	U_{14}–M_{15}	3 601	
20	41 068	24 721	10	61 560	34 040	62 490	0.27	0.016	0.65	0.026	U_{12}–M_{13}	3 601	
21	44 064	23 612	10	60 040	35 640	56 840	0.26	0.012	0.63	0.030	U_{19}–U_{20}	3 601	
22	41 088	32 395	10	64 040	35 400	63 120	0.24	0.028	0.60	0.031	U_{18}–U_{13}	3 601	
23	40 105	35 777	10	60 060	35 480	60 820	0.25	0.022	0.50	0.030	U_{18}–U_{19}	3 601	
24	42 108	45 183	30	61 040	35 700	60 320	0.27	0.010	0.59	0.024	U_{15}–U_{16}	3 601	
25	41 061	21 524	10	63 180	35 390	65 180	0.25	0.027	0.57	0.030	U_{17}–U_{18}	3 601	
26	41 084	30 779	10	61 720	35 960	61 700	0.24	0.030	0.65	0.030	U_{16}–U_{17}	3 601	
27	40 067	22 823	20	59 620	34 240	56 830	0.20	0.012	0.62	0.030	L_{14}–M_{13}	3 601	
28	41 063	22 015	10	62 260	36 240	61 660	0.27	0.026	0.67	0.025	L_{16}–M_{15}	3 601	
29	44 060	21 577	10	65 080	35 260	68 920	0.27	0.029	0.62	0.025	L_{16}–M_{15}	3 601	
30	40 078	26 523	10	58 350	35 180	58 330	0.28	0.019	0.60	0.026	L_{12}–M_{21}	3 603	
31	33 076	.....	20	62 100	40 620	63 760	0.26	0.027	0.63	0.032	U_{22}–M_{13}	3 603	
32	40 106	36 291	30	56 900	35 500	59 960	0.25	0.020	0.58	0.030	L_{22}–L_{23}	3 603	
33	34 079	.....	50	63 940	39 820	62 950	0.30	0.025	0.53	0.025	M_{21}–U_{22}	3 603	
34	34 079	.....	50	63 940	39 820	62 950	0.30	0.025	0.53	0.025	M_{23}–L_{22}	3 603	
35	50 296	166 611	10	60 780	33 660	61 240	0.27	0.011	0.51	0.022	M_5–L_6	3 607	
36	45 235	161 569	30	57 570	31 290	62 240	0.25	0.010	0.60	0.024	U_5–M_3	3 607	
37	45 237	162 279	30	61 740	34 140	59 020	0.26	0.019	0.64	0.030	U_5–M_3	3 607	
38	45 237	162 280	30	61 740	34 140	59 020	0.26	0.019	0.64	0.030	M_5–L_6	3 607	
39	45 234	161 282	20	65 720	35 340	62 510	0.27	0.020	0.60	0.028	L_5–L_6	3 607	
40	45 238	162 782	30	64 120	33 790	62 120	0.24	0.021	0.65	0.034	L_5–L_6	3 607	
41	46 294	169 337	10	62 310	35 860	63 550	0.27	0.010	0.53	0.040	U_5–M_3	3 607	
42	42 286	.....	20	65 520	.....	.....	0.29	0.026	0.50	0.012	M_3–U_4	3 607	
43	42 286	.....	20	65 520	.....	.....	0.29	0.026	0.50	0.012	L_4–L_5	3 607	
44	41 011	3 121	30	61 560	36 150	57 100	0.28	0.021	0.62	0.030	L_5–M_5	3 599	
45	41 011	3 099	30	61 560	36 150	57 100	0.28	0.021	0.62	0.030	U_8–U_9	3 599	Re-test.
46	41 086	31 744	20	57 190	34 950	60 270	0.26	0.012	0.55	0.049	U_{14}–U_{15}	3 601	
47	41 086	31 730	20	57 190	34 950	60 270	0.26	0.012	0.55	0.049	U_{12}–U_{13}	3 601	Re-test.
48	41 063	22 009	40	68 960	36 240	61 680	0.27	0.026	0.65	0.025	L_{14}–M_{13}	3 601	
49	40 066	22 400	00	61 200	34 820	59 500	0.27	0.025	0.68	0.029	L_{14}–M_{13}	3 601	

as described for the main posts. The bottom chords, from L_0 to L_8, were then, by bending, put in a position to correspond to the position of these chords in the truss without stress; the top chords and web members up to L_6-U_6 were then assembled to these chords where necessary. The web members were bent so that when the truss was swung and under dead load their alignment would be straight. After the truss was lined up accurately and made true in every part, all the connections were drilled, as described for the tower posts.

For the cantilever arm, the bottom chord, L_{12} to L_{16}, was assembled to the members, L_{14}-U_{14} and L_{16}-U_{18}, in the deflected position, after which the connections at L_{14} and L_{16} were drilled.

As it was considered undesirable to depend on bolts during erection, the members, L_{14}-U_{14} and L_{16}-U_{18}, were field-spliced near the L_{14} and L_{16} points. After the splices in these members had been properly drilled and tightly bolted up, they were assembled to the bottom chord, as described; and, after the connections to this chord had been drilled, the splices in the members were cut and the short pieces were riveted to the bottom-chord panel points without changing their position from that in which they were reamed or drilled. This did away with any dependence on field bolts during erection, as perfect butt joints were available; it also insured having the connections made in a position which corresponded with the final deflected position of the truss. When coupling up during erection, these members had to be bent; this, however, caused no trouble.

Floor-beam and stringer connections were generally sub-punched and reamed to templates.

Inspection and Tests.

The work at all times, at the mill, the shop, and in the field, was under thorough inspection. All measurements were made by the contractors, and also by the inspector, with steel tapes and other instruments carefully compared with the Railroad Company's standards, which had been checked and corrected by the Bureau of Standards at Washington, D. C. Steel-tape measurements were made under uniform tension by the use of spring balances.

The unusually large riveted connections and splices, built up of many thicknesses of plates and angles, presented difficulties in providing perfectly flat plates of even thickness.

The chief reason that the results, as to the quality of material

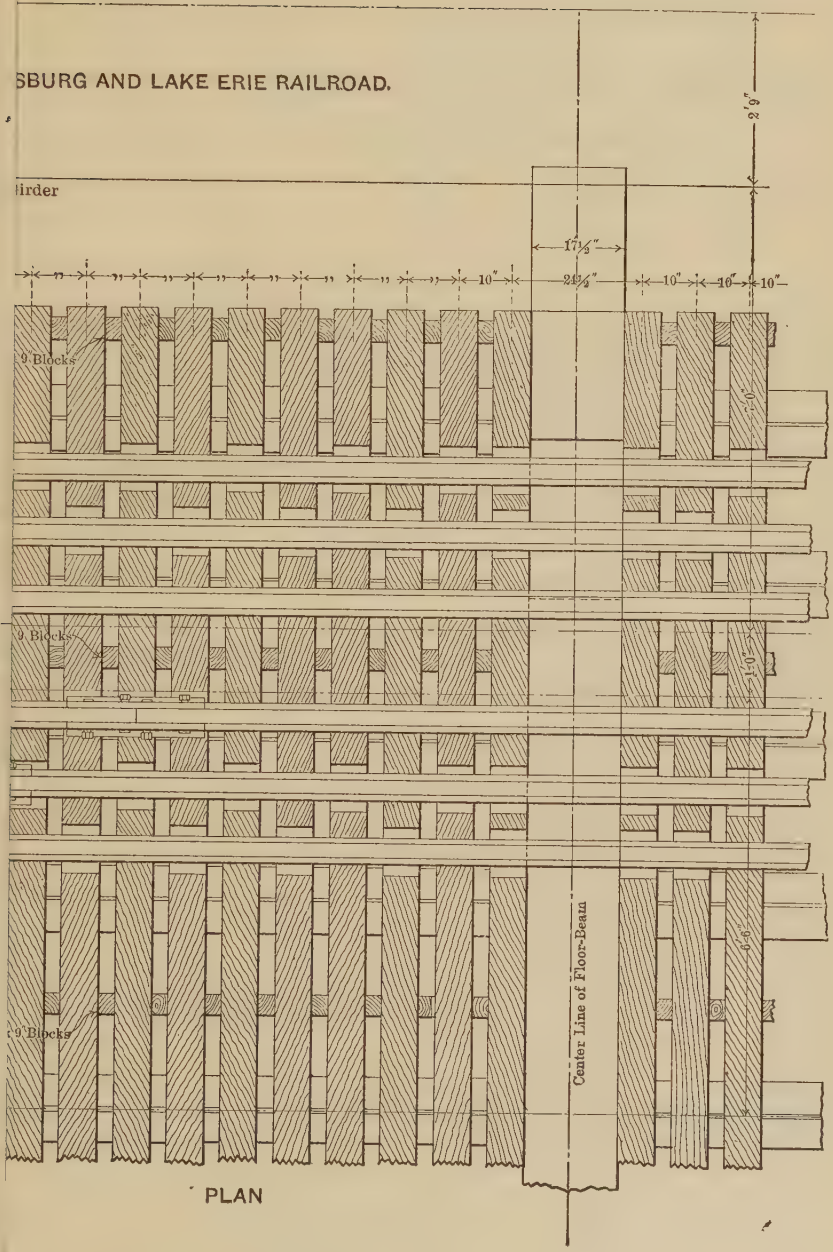
and workmanship, were so satisfactory, apart from the great interest everyone responsible for any part showed in the work, was the fact that the material and the work were in a certain sense inspected before the work was done. This was accomplished by the very efficient manner in which the inspectors discussed in advance all details (and particularly the difficult points to be overcome) with the millmen and the shopmen; in this manner hearty co-operation was secured, which insured satisfactory results.

Some of the joints required flat plates and rolled shapes to be built up to a thickness of 7¼ in. Any variations in the thickness of these plates or in their flatness would have caused serious trouble.

The work of driving 1-in. rivets through this thickness of material, in the shop and in the field, so that the rivets would be tight, also required much study and care by the contractors as well as by the inspectors. The results secured were entirely satisfactory. The best shape for long rivets in the stock near the head was determined by much experimental driving. Milling and boring were done by taking a rough cut first, then the measurements were checked and adjustments made if necessary before making the finishing cut. All pin holes were located very carefully for position and direction. Eye-bar pin holes were bored with two cuts. The clearance between the pins and the pin holes, for pins from 12 to 16 in. in diameter, was required to be $\frac{1}{32}$ in., and the variations were limited to from 0.03 to 0.04 in. In order to meet this narrow margin, every cutter was carefully gauged and recorded, and every pin hole cut by each cutter was measured and recorded. It was found that the holes from 12 to 16 in. in diameter, made by all cutters, were uniformly from 0.004 to 0.005 in. larger than the size of the cutter. After this was established there was very little trouble in cutting the pin holes to the exact diameter desired. The cutters had to be changed frequently, on account of wear. A few pin holes had been made before the variations in size between the pin and the hole had been determined; some of these, with variations of 0.024 in., caused considerable trouble during erection, as the pins drove exceedingly hard.

All material was tested, and many tests were made for compression. Tests were also made on rollers, on sections 1 in. long.

Full-sized compression members were not tested. Full-sized eye-bars were tested, as required by the specifications. Table 1, Plate VIII, gives the results of these tests.


SBURG AND LAKE ERIE RAILROAD.
Girder
2'9"
17½"
10"
21¼"
10"
10"
10"
9 Blocks
9 Blocks
9 Blocks
Center Line of Floor-Beam
PLAN

The annealing of eye-bars and steel castings received much attention; the eye-bar tests show great uniformity, and the thorough annealing of the steel castings made the steel in them very ductile.

Eye-bars were tested for elastic limit, ultimate strength, stretch, and reduction. The stretch of the bar due to the bedding of the pin against the eye was noted; and, although it could not be correctly measured, it was found to be generally as much as $\frac{1}{8}$ in. for each head when the elastic limit was reached.

Rolled-steel coupons, 1 in. in diameter and 1 in. long, taken from eye-bar stock, showed a yield point in compression of from 27 000 to 29 000 lb. per sq. in., at which time the first permanent set of from 0.001 to 0.002 in. could be observed. The yield increased to 15% at 100 000-lb. pressure. After the 100 000-lb. pressure was passed, the steel flowed, the test pieces swelling in diameter under the slowly increasing pressure until, at 250 000 lb., the original height of 1 in. was reduced 50% and the original diameter of 1 in. was increased to $1\frac{7}{16}$ in. This pressure did not develop any distortions or surface cracks.

The 12-in. roller sections were tested in pieces 1 in. thick cut from the ends of the rollers. This test was made to develop the safe bearing load per linear inch of the 12-in. rollers under the tower posts. The loads applied to specimens of this size varied from 10 000 to 16 000 lb. by 1 000-lb. increments. No set could be detected, after these loads had been applied in turn; at 25 000 lb., one specimen showed a set of 0.006 in.; at 32 000 lb., another showed a set of 0.007 in.; at 50 000 lb., another showed a set of 0.032 in.

The first two tests given were made on a compression testing machine, and were satisfactory; the last one was made on a tension machine, which did not work properly.

The maximum loading the 12-in. rollers have to support in the bridge is less than 6 000 lb. per lin. in.

In summing up the results of the tests, it appears that rolled medium steel, of 66 000 lb. ultimate strength and 35 000 lb. elastic limit in tension, has a yield point in compression of from 27 000 to 29 000 lb.; this steel sets 15% under a compression of 100 000 lb. per sq. in.

Cast steel of from 65 000 to 70 000 lb. ultimate strength and 35 000 lb. elastic limit in tension has a yield point in compression of about 13 000 lb., and sets 16% under 100 000 lb. on 1-in. cylinders. Manganese-bronze of 80 000 lb. ultimate strength and from 41 000

to 43 000 lb. elastic limit in tension, has a yield point in compression at 28 000 lb. per sq. in., and a set of from 9½ to 10% under a load of 100 000 lb.

Riveting.

The material was punched ⅛ in. smaller and reamed $\frac{1}{16}$ in. larger than the nominal diameter of the rivet, with the exception of laterals, skid girders, lacing, tie-plates, and diaphragms, in which members full-sized punching was allowed; material more than ¾ in. thick, in tension, and more than ⅞ in. thick, in compression, was drilled from the solid. No punching was allowed in the big connection plates in the trusses. When assembling the joints, made up of many thicknesses of plates and angles, the material was cleaned and painted with much care, the paint used on the surfaces in contact being made thin in order that the thickness might not be increased appreciably. In the trusses of the anchor and the cantilever arms, in the floor-beams, and also in the connections between stringers and floor-beams, generally, 1-in. rivets were used, and ⅞-in. rivets in the other parts. All 1-in. rivets with more than 4-in. grip were tapered, these rivets having a diameter of $1\frac{3}{16}$ in. under the shop head and 1 in. at the point. The tapered rivets had to be forced into the holes with a slight pressure. They filled the holes admirably, and gave excellent results in the shop and in the field. The largest grip used was 7½ in. The number of loose field rivets was less than 2 per cent. The field rivets were driven with pneumatic hammers. Numerous tests showed that straight 1-in. rivets, with a grip of 4½ in. or more, bent sidewise in the hole about 4 in. from the unfinished head, and, after coming to a bearing on the walls of the rivet hole, all upsetting below this point ceased.

Tapered rivets stood up straight and filled the holes. The points of all long rivets were dipped in water before driving. It was found necessary to have the diameter of the rivets not greater than $1\frac{3}{16}$ in. at the base of the taper. For this reason the dies for making these rivets had to be renewed frequently. If the rivets were too large, the walls of the rivet hole would scrape the metal back, and a fin would be formed between the shop head and the member, making it impossible to get the rivet tight.

Skid Girders.

The skid girders, two on each side of the bridge, are unique and of unusual interest, in forming, with the special floor design, a

FIG. 1.—ERECTION OF TRAVELER.

FIG. 2.—ERECTION OF ANCHOR ARM.

remarkable protection against a catastrophe in case of a derailment. There are two horizontal skid girders for each truss, one vertically over the other, 4 and 6 ft., respectively, above the top of the ties. The chords on the track side of the trusses are made up of 6 by 6 by ½-in. angles and 12¼ by ½-in. web plates. The outside chords have 6 by 6 by ½-in. angles and 10¼ by ½-in. plates. These chords are 5 ft. 4½ in. apart, and are connected by web members made up of 3½ by 3½ by ⅝-in. angles.

The track faces of these skid girders are smooth and continuous, and 7 ft. 0 in. from the center line of the adjacent track. They have expansion joints at the proper locations, with rivets countersunk on the track faces of the chords.

The skid girders are attached to the main-truss members by angles connecting with the skid-girder chord web-plates.

Details.

Especial attention was given to the details of the chord splices and the connections of the web members with the chords, so as to pick up the material gradually and also have the splice material arranged symmetrically as far as was possible.

In the anchor arm the panel points are shop-riveted to the bottom-chord sections. It was decided to be undesirable to locate the splices in the bottom chord of the cantilever arm in this manner, on account of the great weight, for erection purposes, of a chord section combined with a riveted panel point, which weight would be about 86 tons. For this reason, splices were made in the bottom chords on each side of the main panel points, thereby reducing the weight of the heaviest single member to about 49 tons.

The suspended span is hung on independent pins; the upper one is located in the attachment plates of U_{18} joint, vertically below the panel-point pin; the lower one is in the attachment plates connecting the truss members at the end of the suspended span, and directly above their intersection point, and at the same time not in the way of ample floor-beam connections. This arrangement simplified the erection problems and permitted the use of a very satisfactory development of bushing around the pins in the suspension hangers so as to reduce the pin-bearing stresses to values suitable for the motion of rotation under loads.

The suspension members are built up of stiff sections, and are proportioned for the loads in tension as well as for bending stresses caused by friction assumed at 25% on the pins resisting rotation.

The pin holes in these members are provided with steel bushings and manganese-bronze linings. Eye-bars were considered, but were not used, as the estimated effect of the bending stress was too great.

The floor-beams are riveted to the trusses, and the stringers are riveted to the floor-beams only in alternate bays; at all other points they are supported on shelves, with room for expansion at the free ends, so as to limit the secondary stresses in the floor system and the bending of the free ends of the floor-beams due to changes in the length of the chords under live-load stresses.

Floor.

The floor of this bridge was designed to be safe, not only when all wheels are on the rails, but also under any probable condition resulting from a derailment on the bridge.

Four stringers, 2½ ft. from center to center, are placed under each gauntlet, and there is a ninth line of stringers under the center line between the trusses to support the inner ends of the ties. Each gauntlet has four main-track rails and two inner-rail guard-rails, making twelve lines of 100-lb. rails on the bridge floor.

The ties are of 7 by 9-in. white oak, 12½ ft. long, placed on edge and sized to 8½ in. The inner ends of the ties butt on the center line of the center stringers. The ties are 3 in. apart in the clear. This spacing is maintained by using three lines of wooden-block spacers or separators on each half of the floor, to prevent bunching.

The tops of the ties are about 1½ in. above the tops of the floor-beams, and are held about ¼ in. away from them by the use of metal plates, in order to allow cinders to be blown off the floor-beams by passing trains.

Wooden guards are not used, as the modern steel-car construction, with drop doors, has forced their location below a plane through the top of the rails within 5 ft. of the center of the track, thereby defeating the object of their use. The spacing of the ties, formerly secured in an indifferent manner by dapping the guard-rails over the ties, is accomplished much more effectively by the block spacers used.

All the ties and the spacing blocks, oak and pine, respectively, have been treated chemically by immersion in an open tank of Carbolineum

FIG. 1.—ERECTION OF ANCHOR ARM.

FIG. 2.—CREEPER TRAVELER ERECTING CANTILEVER ARM

Avenarius heated to a temperature of 260° Fahr. The construction of the deck is shown on Plate IX. All track rails are supported on double-shoulder tie-plates secured to the ties by screw-spikes, as shown on Figs. 1 and 2. No special expansion joints are used in the rails.

The bridge expansion provision amounts to about 5 in. at the L_0 points and to a total of 8 in. at the ends of the suspended span.

The rails are 33 ft. long. The standard rail splices used allow an extreme movement of $\frac{5}{8}$ in. at each rail joint, which amount is sufficient to accommodate a temperature change of 140° Fahr. The rails were given the proper spacing when laid, with the assumption that the temperature would vary from 20° below to 120° above zero. The rails were then immediately secured to the floor over the "fixed" points of the bridge, and, on the shore side of the abutment, on the tracks approaching the bridge, by the use of efficient anti-creeping devices, as shown on Fig. 3. Between these fixed points the rails are free to travel on the tie-plates. All spikes in the vicinity of the joints are located so that they will not interfere with the free travel of the rails, the assumption being that both the rails and the bridge are provided with sufficient freedom to expand and contract between these fixed points, the rails by distributed expansion points, and the bridge at one or two located points; and that the two movements will be adjusted by the rails moving between the fixed points in the tie-plates. No anti-creeping devices are used on the suspended span.

Wind.

Considerable wind stresses have to be taken care of by the anchor masonry at the L_0 points, where, under the passage of each train, the vertical reactions are zero, and the normal reactions under dead loads alone are not great. For these reasons, it was decided to transfer the wind stresses to the anchor masonry by using wind buffers built into the masonry and securely anchored to it. These buffers engage with the bridge between brackets attached to the bottom of the end floor-beams, bearing plates of manganese-bronze being used between the buffers and the brackets. These buffers allow freedom of motion in the bridge, longitudinally and vertically, but resist all side motion.

The wind bracing, commencing at these buffers, is continuous between the upper as well as the lower chords to the U_{20} point in the upper system and to the L_{18} point in the lower, where it engages the

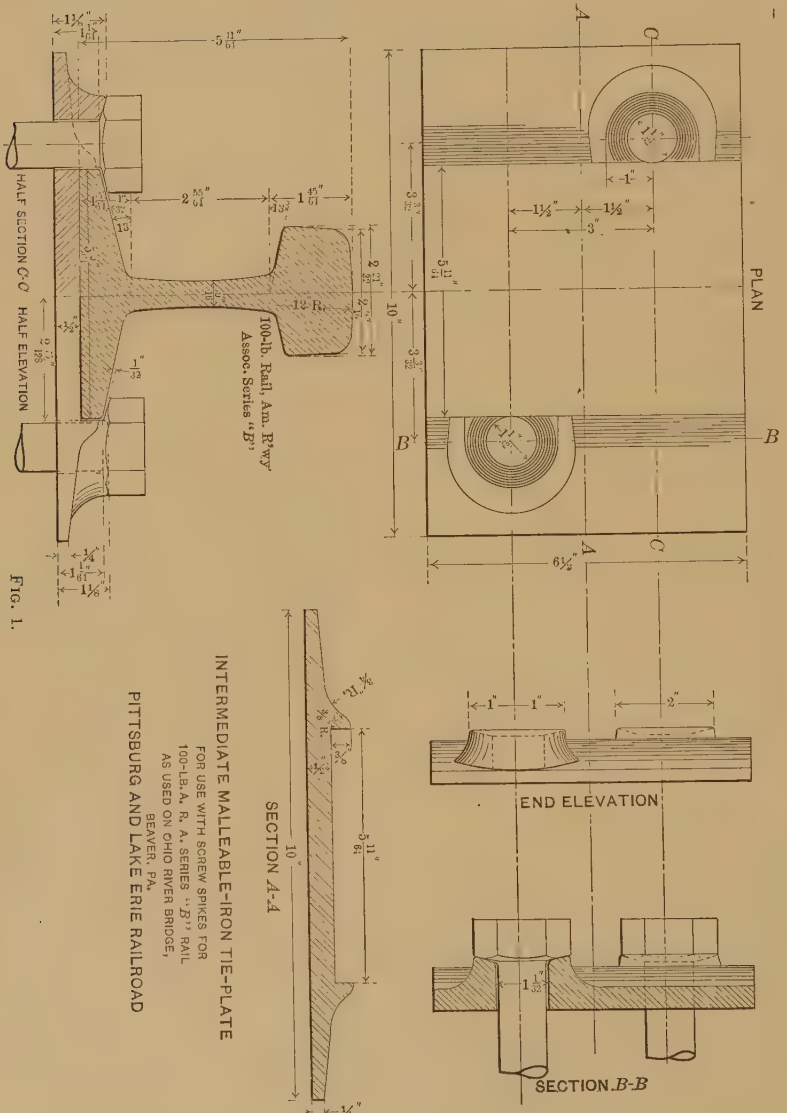

HALF SECTION C-C
HALF ELEVATION
100-lb. Rail, Am. R'wy. Assoc. Series "B"
PLAN
END ELEVATION
SECTION A-A
SECTION B-B
INTERMEDIATE MALLEABLE-IRON TIE-PLATE
FOR USE WITH SCREW SPIKES FOR
100-LB. A. R. A. SERIES "B" RAIL
AS USED ON OHIO RIVER BRIDGE,
BEAVER, PA.
PITTSBURG AND LAKE ERIE RAILROAD

Fig. 1.

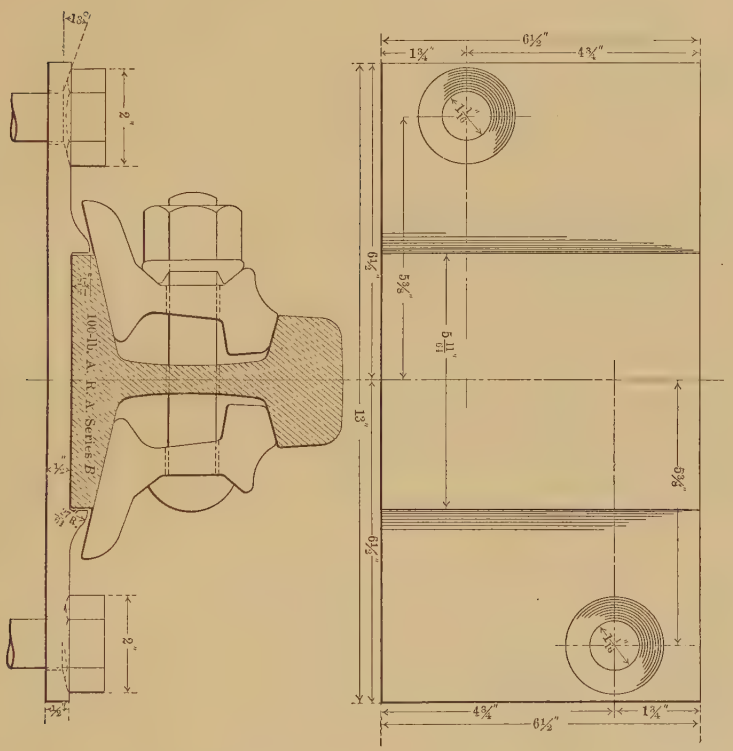
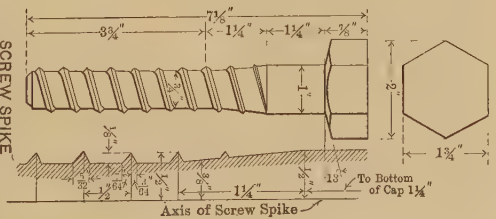


FIG. 2.

MALLEABLE-IRON JOINT TIE-PLATE
FOR 100-LB. A. R. A. SERIES "B" RAIL
AS USED ON OHIO RIVER BRIDGE,
BEAVER, PA.
PITTSBURG AND LAKE ERIE RAILROAD

SCREW SPIKE
To be used with plates shown on this drawing and on Fig. 1.



portal strut of the suspended span and its end floor-beam, respectively, at their middle points, by pins and castings sliding between bronze plates in the strut, or between the brackets on the end floor-beam. The wind bracing in the suspended span is complete between these supporting points at its ends. All wind bracing is stiff throughout.

Falsework and Erection.

The falsework was designed to support heavy concentrations of load under the anchor arms, making double bents necessary under the main panel points, and was built on pile foundations, the piles being driven to refusal by a steam hammer. No settlements occurred in the falsework during erection.

The work of erection was handled very efficiently and expeditiously by the contractor. An outside gantry traveler was used for all material erected on the falsework; this included the fixed span, the anchor arms, and a sufficient portion of the cantilever arms beyond the main piers to allow the erection of the main towers, the inclined posts, and the first two-panel section of the lower chords of the cantilever arms, as shown by Figs. 1 and 2, Plate X, and Fig. 1, Plate XI. The remaining portions of the cantilever arms and the suspended span were erected with a creeper traveler (as shown by Fig. 2, Plate XI, and Figs. 1 and 2, Plate XII), or a traveling derrick, attached to the top-chord panel points when in operation, and moved ahead on ways attached to the same points, which points were designed to serve this purpose.

The creeper traveler was very complete in all its details, its weight was counterbalanced while moving it from panel to panel, and it was handled with great certainty and promptness. The erection was commenced at the south or Monaca abutment by placing the anchor eyebars and the L_0 steel castings. The floor-beams and stringers were then put in place to receive the material tracks. This work was continued to the L_{10} point, where the cast-steel tower-post bearings and rollers were put in place.

The "L_{10} stubs" were then put in position on the cast-steel bearings. These stubs consisted of the lower sections of the posts, with strut sections of the lower chords and the L_{10} panel point, with all its plates and angles shop-riveted. This work was done with the gantry traveler. The traveler was then moved toward the abutment, setting the lower-chord sections, and attaching them to the floor-beams as they were put

FIG. 1.—ERECTING CANTILEVER ARM WITH CREEPER TRAVELER.

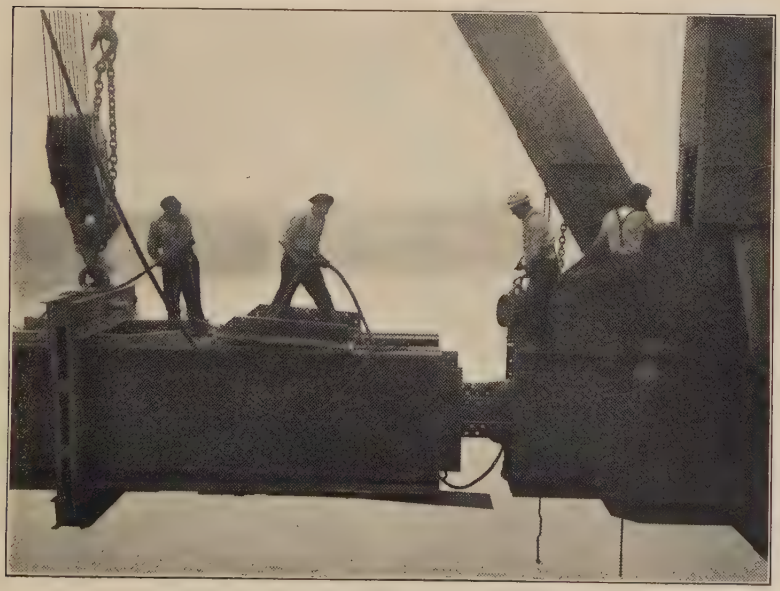
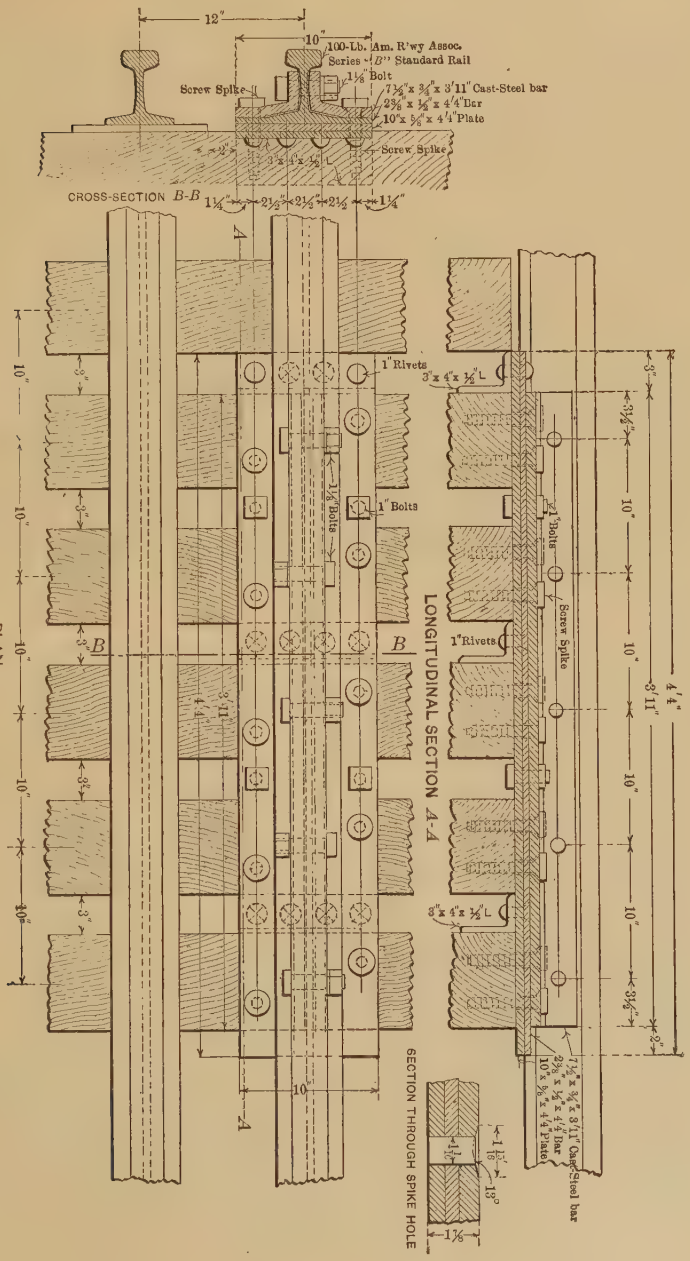


FIG. 2.—ERECTION OF CANTILEVER ARM.



DESIGN FOR ANTI-CREEPING RAIL DEVICE, PITTSBURG AND LAKE ERIE RAILROAD

Fig. 8.

in place. This being completed at the L_0 point, the erection of the trusses was commenced at the same point, and the work was finished as the traveler was moved from point to point to the main-tower posts, where it was placed in a practically central position over the pier. In this position the towers were erected, the upper sections of the U_{10} posts being put in place by stiff-leg derricks placed on top of the gantry traveler. With the derricks in this position, the towers were completed, the upper chords and the first double-panel sections of the cantilever arm were put in place, and all connections for extending the cantilever arm were coupled in the completed L_{12} and U_{12} points. These derricks were then used in erecting the creeper traveler in place on the completed top chords between the U_{11} and U_{12} points. The gantry traveler was practically built in the structure as the tower sections were completed up through it, making it necessary to take the traveler down in order to remove it. When this was done, the traveler was moved to the Beaver end of the bridge, and used in the erection of the 370-ft. fixed span, as shown by Fig. 1, Plate XIII, after which it was moved forward and used on the Beaver anchor arm erection, the same as at the Monaca end.

The material erected by the gantry traveler was delivered by cars; that erected by the creeper traveler was delivered by barges on the river (Fig. 2, Plate XIII), and hoisted directly from them. The material was loaded on the barges at the contractors' bridge plant, at Rankin, Pa., about 35 miles above the bridge site.

The adjusting device used during erection, in making the final connections between the two halves of the suspended span, consisted of wedges operated by large screws in the bottom chords at the L_{18} points, and toggle joints in the top chords, also operated by screws. These wedges and toggles were set so that each half of the suspended span was erected and held in a position a little forward and upward from its final location, making the distance between the erected ends of the top chords on each side of the center of the bridge a little in excess of the length of the closure section. The bottom chord points were a little closer together than their final position, which allowed the eye-bars to be coupled and the pins driven with the center panel point a little below the line of the bottom chords. After the closure sections of the top chords were in position, the bottom chords connected up, and the web members secured in place, the wedges and the toggles were

FIG. 1.—ERECTING FIXED SPAN WITH TRAVELER.

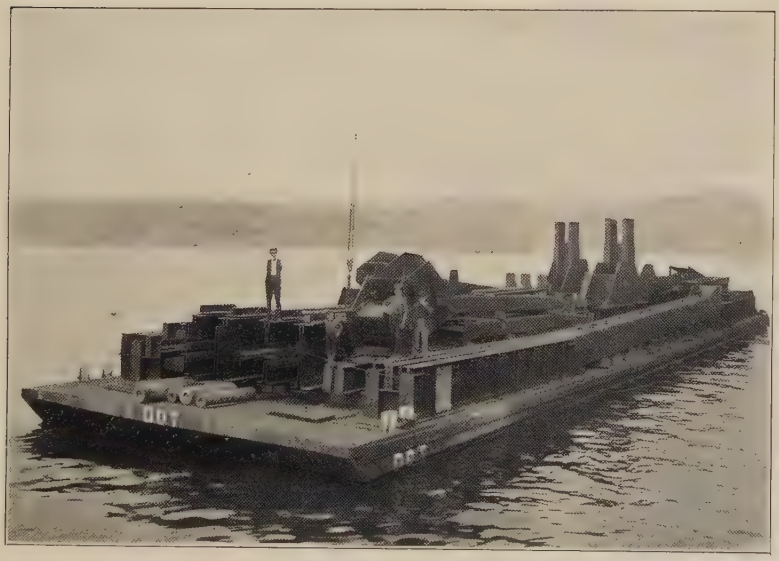


FIG. 2.—BRIDGE MATERIAL DELIVERED ON BARGES.

operated, and, as the two parts of the suspended span came into position, the chord splices were connected, thereby completing the span. This closure is shown on Plate XIV.

The mode of erecting the bridge, and the devices, derricks, and tools necessary for erection and adjustment, also the detail of the falsework under the anchor arms and the fixed span were left to the choice of the contractor, subject to the approval of the Railroad Company as to their general sufficiency and strength.

WEIGHTS.

Steel anchors	399 368 lb.
Trusses, floors, and skid girders	31 441 211 "
Cast-steel work, including main-tower rollers	526 211 "
Total	32 366 790 lb.=16 184 tons.

The 370-ft. fixed span included in the foregoing figures weighs 2 206 tons, and the suspended span, also included in the total, weighs 1 596 tons. The dead load in the wooden deck and rails amounts to 900 tons.

SPECIAL LOADING ON THE BRIDGE USED TO ADJUST THE WEDGES AT THE L_0 POINTS.

After the completion of the bridge, and before it was put in service, a special loading was placed on it in order to develop stresses in the anchor eye-bars in excess of the working loads, and thereby allow the wedges at the L_0 points to be set up so as to maintain this maximum tension in these eye-bars. For this purpose, eight locomotives and twenty loaded cars were made up in two trains, one for each track, each consisting of five cars, four locomotives, and five cars, weighing as follows, for each track:

5 cars	749 100 lb.
4 locomotives	1 344 000 "
5 cars	749 100 "

These trains were placed side by side, with the forward ends at the tower pier adjacent to the anchorage to be adjusted, the trains being entirely on the cantilever arm and the suspended span.

Elevations were taken at each floor-beam before, during, and after the application of the load, and the "travel" of the U_{10} point was noted

by using a transit to plumb down to the masonry from that point. The details of the measurements are given in Table 2.

TABLE 2.—Elevations of Floor-Beams on Ohio River Bridge During Test, May 10th, 1910.

Point read.	West Side.				East Side.			
	Before loading.	During load.	Difference.	After load removed.	Before loading.	During load.	Difference.	After load removed.
North Span (Load on North Cantilever Arm and Suspended Span).								
L 0	755.659	755.695	—0.036	755.667	755.653	755.669	—0.016	755.675
L 4	756.043	756.115	—0.072	756.049	756.038	756.109	—0.071	756.050
L 6	756.176	756.245	—0.069	756.181	756.163	756.234	—0.071	756.170
L 10	756.405	756.397	—0.006	756.401	756.398	756.397	—0.001	756.396
L 14	756.455	756.262	—0.193	756.444	756.460	756.272	—0.188	756.448
L 16	756.457	756.155	—0.302	756.441	756.460	756.160	—0.300	756.435
L 18	756.430	756.028	—0.402	756.409	756.435	756.016	—0.419	756.407
L 23	756.416	755.986	—0.430	756.388	756.412	755.953	—0.459	756.383
South Span (Load on South Cantilever Arm and Suspended Span).								
L 23	756.411	755.955	—0.456	756.396	756.423	755.956	—0.467	756.392
L 18	756.451	756.031	—0.420	756.444	756.456	756.043	—0.413	756.428
L 16	756.461	756.148	—0.313	756.457	756.475	756.180	—0.295	756.460
L 14	756.481	756.263	—0.218	756.475	756.479	756.292	—0.187	756.465
L 10	.....	756.397	.....	756.403	.....	756.412	.....	756.410
L 6	756.197	756.221	—0.024	756.194	756.203	756.261	—0.058	756.200
L 4	756.065	756.088	—0.023	756.064	756.064	756.121	—0.057	756.065
L 0	755.679	755.688	—0.009	755.680	755.676	755.676	—0.000	755.674
North Span (Load on Anchor Span).								
L 0	755.659	755.666	—0.007	755.667	755.653	755.654	—0.001	755.675
L 4	756.043	755.974	—0.069	756.049	756.038	755.981	—0.057	756.050
L 6	756.176	756.116	—0.060	756.181	756.163	756.111	—0.052	756.170
L 10	756.405	756.389	—0.016	756.401	756.398	756.389	—0.009	756.396
L 14	756.455	756.456	—0.001	756.444	756.460	756.464	—0.004	756.448
L 16	756.457	756.462	—0.005	756.441	756.460	756.464	—0.004	756.435
L 18	756.430	756.449	—0.019	756.409	756.435	756.441	—0.006	756.407

Top of North Tower moved 0.183 ft. S. under load.
" " South " " 0.150 " N. " "
" " North " " 0.030 " N. " " on Anchor Span.

The deflection of the bridge and the movement of the U_{10} points corresponded almost exactly with the estimated amounts. The rollers at the L_{10} point also responded to the deflection in the lower chord, at the ratio of the radius of the roller bearing to the height of the tower.

Under the auspices of Mr. Howard, of the Bureau of Standards, of Washington, D. C., extensive measurements are being made with instruments of precision between points drilled in the material. These

FIG. 1.—CLOSURE OF SUSPENDED SPAN.

FIG. 2.—CLOSURE OF SUSPENDED SPAN.

points are approximately 20 in. apart, and measurements were made while the members were under no stress before erection. Similar measurements are being made now—after erection—to determine, by the variations in length between the holes, the amount of dead-load stress in the members. These measurements are precise enough to determine variations of from 150 to 200 lb. per sq. in., and it is hoped that a complete report of them will be made by the Bureau of Standards in the near future. By this method it has been determined that the dead-load stresses actually existing in the lower chords at each side of the main-tower posts are, on an average, 5 800 lb. for the L_8-L_{10} members and 5 700 lb. for the L_{10}-L_{12} members, and that the total stresses under the live loading noted amounted to 10 000 lb. per sq. in. in the same members.

This bridge was constructed for The Pittsburg and Lake Erie Railroad Company, Col. J. M. Schoonmaker, Vice-President, J. A. Atwood, M. Am. Soc. C. E., Chief Engineer, A. R. Raymer, M. Am. Soc. C. E., Assistant Chief Engineer, and Albert Lucius, M. Am. Soc. C. E., Consulting Bridge Engineer and Inspector.

It was built by the McClintic-Marshall Construction Company, of Pittsburg, Pa., Paul L. Wölfel, M. Am. Soc. C. E., Chief Engineer, and was completed in May, 1910.

The writer is under obligations to Messrs. Lucius and Wölfel for valuable assistance in the preparation of this paper.

DISCUSSION

Mr. Hudson.

C. W. HUDSON, M. AM. SOC. C. E. (by letter).—The form of truss adopted for this bridge is typical of American practice. It is possible, however, that the form has been too closely adhered to, and that many of its supposed advantages are illusory.

Where the requirements of grade and clearance demand a floor of small depth, the conditions are usually met, as in this case, by some form of subdivided truss. This introduces in the same main panel of the cantilever arm a vertical member, which receives its maximum live load from the floor-beam at its foot, and a horizontal member, which is stressed to the maximum either with or without the load at the foot of the sub-vertical. The sub-vertical cannot be cambered for both conditions of maximum stress in the chord, and, therefore, the chord is subject to considerable transverse load in addition to its direct stress. As they carry only one panel load, the sub-verticals are small members and cannot properly hold all parts of the chord against column failure, as they are generally assumed to do.

Those sub-members which are not subject to primary stress and are intended only to shorten the unsupported lengths of compression members, sometimes pull or push the latter seriously out of line, and, as they are small in section and long, they do not accomplish properly the duty for which they are intended.

The size of the sub-verticals, with reference to the main post and main diagonals at the main piers, is clearly shown on Plate V, while the size of the bottom chords is not shown in direct relation to the sub-vertical. It can be stated that while they are large enough to hold the chords in a definite position in a vertical plane, they cannot fully hold them from twisting and failure about their plane of least resistance to flexure. The theory of flexure, and tests on full-sized columns, show that the plane of least resistance to flexure is not in general that of either of the principal axes of the member.

Careful study of the displacements which the panel points of a subdivided truss must undergo under the action of live load raises serious question as to the usefulness of sub-members, and for trusses of great height and relatively short main-panel length, they may be a source of danger.

It is believed that a long truss should have a long panel length which should be nearly equal to the truss depth at any point, in order that secondary stresses may be small.

There is no question but that the floor may be kept shallow in a long panel, and that long panels may be erected by the use of temporary members.

For short spans, the pin-connected truss with tension members of eye-bars has been driven out of use by a better truss having riveted

joints and tension members of such form that their component parts are made to act as a unit and to permit of making suitable connection between the members meeting at a point. Mr. Hudson.

It is believed by many engineers that the riveted truss for long spans is decidedly superior to the eye-bar, pin-connected form, although this has not been completely proven by the results of experience.

This structure may be, and undoubtedly is, safe in every way, but the same result could have been accomplished better by a riveted truss without sub-members.

HENRY S. PRICHARD, M. AM. SOC. C. E. (by letter).—The Ohio River Railroad Bridge at Beaver, Pa., is one of the great bridges of the world. It is notable not only for its size, capacity, and safety, but for the foresight, care, and skill with which it was designed and constructed; the originality and ingenuity by which some of the problems presented were mastered; and the complete success which rewarded the work of the engineers who planned it and supervised its fabrication and erection. Mr. Prichard.

In view of the importance of the bridge and the ever-increasing loads on American railways, it was wise to provide for a load of 6 000 lb. per lin. ft. of each track, and corresponding engines; and in view of this heavy load and of the daring, not to say hazardous, unit stresses permitted in some large bridges, it is refreshing, and tends to restore confidence in American bridge engineering, to have the author refer to the limiting stresses for combined dead load, live load, and ample impact—16 000 lb. per sq. in. in tension and 14 000 lb. in compression —as high.

The regard for the life and safety of the bridge displayed in the choice of loads and unit stresses was consistently carried into the design of the details and the supervision of the shopwork and erection.

The horizontal girders which would protect the bridge and guard against accident in case of derailment, by compelling the train to skid over the ties instead of bumping into the trusses, and the extra stringers to support the ties in case the train should skid, will probably never be needed, but if they ever should be, they will be very much needed. They constitute an insurance against accident to life and property of a very satisfactory kind, which substitutes prevention for compensation, and is well worth all it cost.

The radial roller bearings under the towers at the ends of the channel span solve in a very satisfactory way the problem presented by secondary stresses in chords and unequal pressure on masonry by practically avoiding them. This detail was expensive, but its cost was a mere bagatelle in comparison with the total, and it may prolong the life of the bridge many years and, in the meantime, will contribute to its reliability.

In this connection it is well to point out that the enormous load on

Mr. Prichard.

the channel piers, by reason of the problem presented of designing satisfactory shoes and details, is doubtless responsible for the tall tower posts which divert to themselves a considerable portion of the load which otherwise would travel down the diagonal end posts, as a straight top chord between the ends of the diagonal end posts would have been more economical as regards the weight of the main section of the trusses.

The problem of secondary stresses induced by the endeavor of the floor system to take part of the chord stresses has been avoided by providing for expansion in rail joints and in connections of stringers to floor-beams. This problem is a serious one in long-span bridges, and is often neglected, with the result that serious horizontal bending stresses are developed in the flanges of the floor-beams, and tension in the rivets connecting the stringers to the floor-beams.

The author calls attention to the fact that:

"The dead-load stresses were computed from weights figured from the stress sheets. This method was repeated until the sections, computed on the basis of the actual dead load, figured from the shop drawings, came within 2% of the sections used."

This careful and conscientious method should be the general practice in designing long-span bridges (in which the dead load is such a large proportion of the total), and not a matter for special commendation; but prominent cases in which it was not followed, and in which the dead-load stresses were greatly under-estimated, are yet fresh in the memories of engineers.

The study which was given in advance to difficulties likely to arise in building and connecting such massive members, and the efficient manner, of which the author makes mention, in which the inspectors discussed such difficulties in advance with millmen and shopmen, are especially to be commended. It is hoped that the author will describe some of these difficulties and the efficient manner in which they were overcome.

The success achieved in anticipating and providing in advance for the difficulties to be encountered in carrying out the plans, and the precision with which the work went together and maintained its proper levels and alignment during erection, were due to the large range of pertinent talent in the employ of the Railroad and the Contractor, and the broad-mindedness of the Chief Engineer and Assistant Chief Engineer of the Railroad, who exemplified the difference between self-confidence and self-sufficiency by deriving the full benefit of the talent at their command, by keeping in touch and discussing important points with other members of their profession, and by keeping in their own hands the final decision of all important matters.

The bridge marks an advance in bridge engineering, and should be a precedent for other large structures.

ALBERT B. HAGER, ASSOC. M. AM. SOC. C. E. (by letter).—There are three features connected with the design and fabrication of this bridge which can well be emphasized, as they may otherwise be lost sight of in the many more impressive facts involved in the execution of this project. The thorough manner in which these three features were taken care of shows clearly that the Railroad, the Consulting Engineer, and the Contractor, all had a proper appreciation of their great importance. Mr. Hager.

These features are:

(1) The care exercised in determining the true dead load from which the dead-load stresses were computed;

(2) The fact that the structure was practically assembled in place at the shop, instead of depending largely on reaming to templets for the accuracy of shopwork and the fit of field connections;

(3) "The fact that the material and the work were in a certain sense inspected before the work was done."

(1).—It would hardly seem necessary to call attention to the importance of determining the true dead loads for long and large structures in which the dead-load stresses form such a large proportion of the total stresses. For these comparatively unusual structures, the data, ordinarily at hand for predicating the dead load with reasonable accuracy, do not exist, and therefore the true dead load must be determined by successive approximations based on the actual design.

It is unfortunately true, however, that some large bridges have been built without checking up the assumed dead load properly, and subsequent investigations have shown that these assumed dead loads were considerably below the actual weight of the completed structures. It will be remembered that the Royal Commission which investigated the causes of the collapse of the Quebec Bridge found that the dead load used in the design under-ran the weight of the finished structure by an amount sufficient to have required the condemnation of the bridge, if completed as designed. The writer has participated in the investigation of the strength of two of our largest bridges. In the first of these the purpose of the investigation was to determine whether the live load could be increased without rendering the structure unsafe, and it was found that this could not be done—not because of the increased live-load stresses, but because the dead-load stresses, computed from the weight of the completed structure, exceeded those used in the original design by nearly one-third. In the second of these investigations, it was found that under-estimating the dead load was one of three contributing causes to the weakness of the structure under the specified live loads.

Mr. Hager.

The dead-load stresses in bridges of unusual size or construction should be re-computed after the completion of the working drawings, and the sections revised if necessary, and no shopwork should be begun until this has been done.

(2).—The practice of reaming field connections to templets, which has been generally used by fabricating companies with ordinary structures, is one which the writer believes to be decreasing, and one which will decrease still more. The efficiency of these templets depends as much on the accuracy of their setting, as on their accuracy of construction, and this requires supervision of a grade not generally to be obtained at a bridge shop.

The inspector cannot check up the fit of field connections, on the finished members, with the same accuracy as is possible in laying out the unassembled material. The cost of assembling structures complete at the shop may be slightly greater than that of reaming the field connections to templets, but the increased certainty of easy erection and freedom from back charges, for correcting shop errors in the field, should go far toward offsetting this. The perfection with which the members of this structure went together in the field is undoubtedly due in large measure to the fact that they had already been assembled in place at the shop. Many, but not all, of our large structures have been thus assembled, and this practice should be made general.

(3).—A completed steel member which has not been properly fabricated is at best a difficult thing to repair so that it will satisfactorily serve its purpose, and very frequently it cannot be repaired. At the same time, these unsatisfactory members represent a greater or less amount of the contractor's money, and their unqualified rejection works a very considerable (even though deserved) hardship on him. The problem of where to draw the line in this question, and when to say that a member can be satisfactorily repaired and when not, is the hardest one that the inspector has to face, but it is one that is continually being presented to him. Inspectors, as a class, are not men of technical education or experience in other departments of the work than their own, and yet are subject to the temptation to decide matters, involving considerations with the importance of which they are not familiar, without referring these questions to the proper authorities, because of the delay which this would involve, and the pressure they are under not to hold up the progress of the work.

For these reasons, the writer believes that the most important part of shop inspection is not, as is frequently assumed, the checking up of the dimensions and connections of the completed material, but the inspection which Mr. Raymer refers to as taking place before the work is done. A careful study, by the inspector, of the methods in vogue at the particular shop in question, and the talking over of the difficult

points with the shop superintendent and foremen, will usually secure work of much better quality, and with much less friction, than can otherwise be done. This will also tend to remove the feeling of antagonism which the shopmen frequently have for the inspector. Mr. Hager.

The same reasoning applies to the inspection of the material at the mills, and its more general application would lead to securing better material and more efficient inspection.

F. W. Skinner, M. Am. Soc. C. E.—The essential points have been presented so ably and so clearly that an attempt to add to the discussion is difficult; but, either because of the brevity of the paper or the author's modesty, some points have been overlooked, and it is only just to those responsible for the design and its execution to refer to them. Mr. Skinner.

The speaker made a close examination of the design and of the structure during erection. The point which impressed him most was the manner in which the design and construction was treated as the problem of a great bridge from the outset—as an inherently great structure, with commensurate original conception, and design in a class by itself—and not as the enlargement of a small structure and the adoption of standardized details tortured to suit, or to attempt to suit, the conditions. The work was done on a large scale throughout, and was consistent.

Another point was the logical sequence of the operations in the computing, designing, and drafting rooms. The important details of the structure—its main connections—were first designed, and they essentially determined the members.

Some of the details are important, and differ from those of other great structures. They show the latest ideas in the development of great bridges. Other details conform to advanced standard practice, thus realizing the best principles of design by accepting the good, rejecting the bad, and modifying and creating, where necessary, to suit special conditions.

The connection of the deep bottom-chord webs by full-length longitudinal latticed diaphragms and specially fitted transverse diaphragms insured the development of their compressive strength, and prevented distortion during construction.

Unusual care and ingenuity were exercised in designing for the great chord stresses efficient splices which could be properly field riveted. The gusset-plates at the lower chord connections are of such unusual size that they not only reached but exceeded the limits of rolling and shipping, and were in some cases spliced. Some of the finished members also about reached the clearance limits of shipment, and required special provision for transportation.

It is interesting to note that the maximum sectional area of the

Mr. Skinner. bottom chord is nearly the same as that of the first Quebec Bridge, of more than double this span, which was also proportioned for two railroad tracks in addition to highway traffic. In order to prevent the possibility of transmitting lower chord stresses through the floor system, the stringers have intermediate expansion bearings, that is, with sliding connections two panels apart.

The accuracy of the shop work was such that allowance was made for the elongation of long plates caused by punching, a refinement of fabrication which was justified by the fact that the separate pieces matched well, and, even in the field, the riveting gangs drove as many as 700 rivets per day.

The erection methods and plant were unusually simple and efficient. The cantilever traveler presented novel and important features in the adjustments to compensate for the varying inclination of its track, and in the counterbalancing and anchoring system.

Costly special clamps, hooks, or yokes, for handling very heavy members, were largely replaced by a convenient and efficient wire-rope sling made with many parts of small, flexible, plow-steel cable rove through large end shackles like a many-part tackle, and with the different parts clamped together under equalized tension.

Mr. Carpenter. A. W. Carpenter, M. Am. Soc. C. E. (by letter).—It was the writer's good fortune to be in touch with Mr. Raymer during the construction of this bridge, and, at his invitation, to visit the work both at the shop and in the field. In design and construction this bridge is certainly of a very high class, and shows what can be done by American engineers and shops at their best. The whole construction is marked by careful and intelligent thought in design and skill in execution.

Mr. Raymer mentions the unit stresses as being rather high. Since the calculated impact allowance for live load appears to be greater than by the well-known Schneider formula, $\frac{300}{L + 300}$, it would seem that the unit stresses would not be considered high by most bridge engineers. It would also seem that the tension unit might have gone higher with safety.

The writer tried to find a reasonably long column, in proportion to lateral dimensions, in this bridge, from examination of the drawings. Without recalling the ratio of length to least radius of gyration of the longest discovered, he found that the columns were braced at such comparatively close intervals that there was hardly any chance for column action to develop.

The only point the writer noted in the design which does not appear fully to take care of the function for which it was intended, is a minor one, and is the breaking of the riveted stringer connections in every other panel. The floor-beams and the stringer connections

in the panels which are riveted must share the stretch or compression of the bottom chords, and the floor-beams possibly suffer more bending than if the stringers were riveted continuously. Mr. Carpenter

The provision for expansion and contraction in the rails, without special expansion joints, is clever and simple, but such an arrangement would not be adapted to rails embedded in a pavement.

The dead-load strain measurements inaugurated on this bridge add to the progressive character of the structure. If these measurements prove successful, there need be no question of the stresses in this structure, as they will not have to be determined by calculation from assumed loads and truss outlines, but can be measured directly on the metal. The live-load strains, of course, can be measured by extensometers on any structure, but the dead-load strains cannot, unless the points of measurement are established before erection. In small bridges the dead-load stresses are not so important, but in structures of the magnitude of this one, in which the dead load is greater than the maximum live load, the dead-load stress is the more important and heretofore has had to be determined solely by calculation.

The measurements have been arranged so as to include secondary strains, variations over different portions of the same member, lattice bars, pin-plates, etc. It is thought not too much to predict that these measurements will be a matter of world-wide interest, and be considered as one of the notable steps in the development of bridge design.

Albert Lucius, M. Am. Soc. C. E. (by letter).—Referring to Mr. Carpenter's criticism that, in his opinion, the breaking of the riveted stringer connections at alternate floor-beam panels does not seem to take care of the function for which it was intended, the writer begs to call his attention to the fact that the purpose of the floor expansion joints is to prevent the bending of the floor-beams, due to the cumulative shortening of the lower chord under compression, at successive panel points. The floor-beams either had to be able to move along with the shortening truss chords, or stay back with the stringers and bend. They could not move without the expansion joints of the stringers, and they could not bend because they were too stiff, having chord flanges 20 in. wide and nine lines of stringers to hold them. It would have been impossible even to assemble the floor-beams to the truss connections, in the field, on falsework, without expansion joints, which are one of the features of the design. Mr. Lucius.

Concerning Mr. Hudson's criticism of the general design of the bridge, it may be true that long panel trusses for double-track and heavy loading can be developed with shallow floor systems, the details of which have no drawbacks of their own, but the writer does not know of them. It is certainly true that simple paneled trusses are preferable to sub-paneled ones where the proportion of the depth of the truss

Mr. Lucius. and the panel length bear a proper relation to each other. It may also be true that the riveted connections of the truss members can be carried to any length, no matter how large the sections of the members which are to be connected, but on this point much experience has yet to be gained, and much experimental and corroborative data must be provided before such an assertion will be generally accepted. When a definite construction comes up for development, some design must be chosen, which will be a compromise between ideals and present conditions, and if such design is consistently carried out and proper provision is made in the details to adjust the members to their functions, the result will be satisfactory; criticism must not be based on generalization alone.

Both Mr. Prichard and Mr. Hager are right in attributing much of the success of the construction to the mutual confidence and agreement between the Company and the Contractor and the open-minded and liberal views of the Company's Engineers, which enabled the representatives of both parties to continue the development of the design to the end, and to co-operate thoroughly during the manufacture of the material and the fabrication and erection of the bridge.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1196

DAMS ON SAND FOUNDATIONS: SOME PRINCIPLES INVOLVED IN THEIR DESIGN, AND THE LAW GOVERNING THE DEPTH OF PENETRATION REQUIRED FOR SHEET-PILING.*

By Arnold C. Koenig, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. W. G. Price, Alexander Potter, T. Kennard Thomson, G. E. P. Smith, Allen Hazen, R. C. Beardsley, and Arnold C. Koenig.

All textbooks on the subject of dams which the writer has had occasion to examine emphatically discourage the idea of attempting to construct permanent dams on any kind of foundation other than solid rock bottom, although most of them contain one or more illustrations of the types which are generally designed for construction on alluvial soils, such as firm beds of clay, or deep strata of cemented gravel (commonly called "hardpan"), or on sand, provided the conditions are such as to permit the construction of a curtain, or cut-off wall, of sheet-piling, or a core-wall of masonry or of clay puddle, so as to penetrate a firm stratum of denser material.

The frequent failures of sheet-pile dams, on the "bottomless" sand beds of Western streams, have served merely as the basis for warnings, and not as incentives to study and investigation, on the part of the authors of the standard books on the subject of dams, which the practising engineer consults for information and precedents.

* Presented at the meeting of March 15th, 1911.

Many crude brush and log dams were constructed, and maintained successfully for long periods of years, on these "bottomless" sand beds, by practical and resourceful pioneers who had no theories or precedents other than the example of the beaver.

The operations of British engineers in contending with sand, during the construction of the large Government irrigation works in India, furnish some expensive precedents for the expenditure of unlimited funds; but, our pioneer Western irrigation surveyors were confronted with the problem of coping with streams having apparently bottomless beds of shifting sands, with funds which were generally limited to the few dollars which the pioneer settlers could borrow on their personal characters and future prospects, because their "desert" homesteads, which now rank among the finest farms in the country, were in those times practically worthless as collateral security.

The writer has spent most of his life in the sandy regions of the West, and has had occasion to seek information on practical methods of dealing with treacherous sand, as a young engineer whose ambitions were involved in the successful solution of such problems; therefore after several years of professional familiarity with it, the term, "quicksand," has less terror for him than it ordinarily arouses.

The following paper is submitted for the consideration of those who have occasion to deal with such problems, not in the sense of establishing authoritative rules, but with the hope of arousing discussion, from which it may be the writer's privilege to learn things which have not been brought out in his own experience.

Seepage, Percolation, and the Movement of Underground Waters.— The velocities of flow which occur in seepage, percolation, or in the forced flow of water through sand, gravel, clay, loam, or other pervious materials, are affected by coefficients of capillary attraction and frictional resistance.

The effects of these forces are co-existent, but not identical, and they vary inversely as the sizes and proportion of the interstices between the particles of the alluvium. In fine clay, therefore, in which the interstices are of infinitesimal size, it is almost impossible to force an appreciable flow of water through a thickness of a few feet. If water finds passage, however, through any point of variable density, or any fissure or shrinkage crack, a barely appreciable trickle, under pressure, will rapidly disintegrate and abrade the clay, and, as the

stream thus increases in volume, will quickly undermine or destroy any structure which may be dependent on the density of the clay for its stability.

In the case of sand or gravel, with relatively large interstices, the forces of friction and capillary attraction offer less resistance, and therefore water may be forced through them with an appreciable velocity of flow.

In any alluvial soil in contact with, or subject to the pressure of, water, a movement of water through it is inevitable, whether it be the infinitesimal progress of seepage through fine clay, percolation through sand, the flow through an embankment of broken stone, or the rush of water through the crevices and open spaces between large boulders, the safe, permissible velocity of flow must be confined to that which the material will stand without the movement, displacement, or abrasion of the particles which compose it.

Turneaure and Russell* refer to numerous experiments which have been carried out at various times to determine the velocity at which water will flow through sands and gravels of different grades of fineness. These experiments were made for various purposes, such as to determine the dimensions of sand filters, the probable supply of water from wells, etc.

D'Arcy, Hagen, Hazen, and others, found that for grades of sand varying from very fine sand to fine gravel, the velocity of flow closely followed the law of flow through capillary tubes.

In considering the flow of seepage, or ground-waters, we are not concerned in refinements of experimental data which indicate the influence of temperature on the rate of flow; therefore, after eliminating these, the result of the experiments is the following formula:

$$V = 8\,200\ d^2\ s.$$

In which V = the velocity of flow, in feet per day;

d = the effective size of the sand grains (varying from 0.1 mm., which is very fine sand, to 3 mm., which is fine gravel); and

s = the slope of the ground-water surface (hydraulic gradient).

According to this formula, the velocity of flow of ground-water,

* In "Public Water Supplies."

through fine sand having an effective size of grains of 0.1 mm. and a general slope of ground-water surface of 5 ft. per mile (hydraulic gradient = 0.001), would be:

$$V = 8\,200\ d^2\ s = 8\,200 \times 0.01 \times 0.001 = 0.082 \text{ ft. per day.}$$

The velocity of flow through gravel having an effective diameter of grains of 3 mm. and a general slope of water surface of approximately 50 ft. per mile (hydraulic gradient = 0.01) by this formula would be:

$$V = 8\,200\ d^2\ s = 8\,200 \times 9 \times 0.01 = 738 \text{ ft. per day.}$$
$$= \text{approximately } 0.0085 \text{ ft. per sec.}$$

The Report of the Massachusetts State Board of Health for 1892 contains the results of a series of experiments to determine the velocities of flow of water which it is possible to obtain through screened gravel, of various grades of fineness, assuming 40% porosity. These results are shown in Table 1.

TABLE 1.—Velocity of Flow of Water, in Feet per Day in Screened Gravel, Assuming 40% Porosity, Based on Experiments of the Massachusetts State Board of Health.

Slope, s.	Effective size, in millimeters.									
	3	5	8	10	15	20	25	30	35	40
0.0005	28	82	164	246	410	656	902	1 230	1 640	2 050
0.001	57	172	335	475	820	1 210	1 680	2 250	3 030	3 690
0.002	115	328	639	902	1 550	2 250	3 030	3 930	4 830	5 820
0.004	221	631	1 230	1 700	2 870	3 930	5 000	6 060	7 130	8 200
0.006	336	918	1 690	2 250	3 690	5 080	6 390	7 620	8 930	10 100
0.008	443	1 160	2 060	2 780	4 340	5 900	7 380	8 930	10 400	11 800
0.01	549	1 410	2 460	3 150	5 000	6 800	8 440	10 000	11 500	.....

From an examination of the figures in Table 1, it becomes evident that all the foregoing experimental data are, at best, only rough approximations. The formula: $V = 8\,200\ d^2\ s$, which was determined from experiments with various grades of sand, gives much larger results than the quantities in Table 1 for screened gravel. To modify the formula so as to produce the results given in that table would involve a variation of the coefficient (8 200) between 6 222 (for $d = 3$ mm. and $s = 0.0005$) to 940 (for $d = 35$ mm. and $s = 0.01$).

For many years the water supply of the Citizens Water Company, of Denver, was secured from about a mile of timber crib, 30 in.

square in section, and about a mile of perforated 30-in. pipe, both submerged in the sands of the Platte River at depths varying from 14 to 22 ft. The timber crib was open at the bottom, so that, with the openings in the cribbing, perhaps one-half of its superficial area was open for the inflow of water, while, for the pipe line, the net area of the perforations would probably bear a much smaller ratio to the circumferential area of the pipe. It may be assumed, therefore, that on the 2 miles of combined crib and pipe line the total area of inlet openings would approximate 25% of the surface area exposed to contact with the water-bearing sand, thus affording a total net area of inlet openings of 26 000 sq. ft. The water was led away through pipes, by natural flow, and the supply thus secured was between 400 000 and 450 000 cu. ft. per day, which is equivalent to 13 cu. ft. per day per square foot of inlet opening, or a velocity of inflow of 0.00015 ft. per sec.

Being submerged under the bed of the river at depths varying from 14 to 22 ft., these cribs are entirely surrounded by water-bearing sand; therefore, as the water may approach radially from every direction, it must be assumed that the maximum velocity of flow through the sand is less than one-fourth of the velocity of inflow through the perforations and openings.

Assuming an average head of 16 ft. as acting on these cribs, the spouting velocity of water entering the pipe through one of the perforations from a free body of water under the pressure due to a 16-ft. head ($V = \sqrt{2\,g\,h}$) would be 32 ft. per sec., while the actual average velocity of inflow is only $\frac{1}{2\,000}$ of such rate.

All formulas for the velocity of flow of water, under special conditions, are modifications of $V = \sqrt{2\,g\,h}$, with coefficients introduced to provide for the special conditions of frictional resistances encountered. In working backward from the results indicated above, so as to determine the special coefficient which must be applied to the formula to adapt it to this special case, we obtain the following:

$$V = 0.00004\ \sqrt{h}.$$

The lack of harmony in the results indicated in all the foregoing data makes it impractical to devise even an approximate formula for the determination of the magnitude of the force produced by the flow of seepage water under a line of sheet-piling. Even if it were possible to determine the relative effects of frictional resistance and capillary

attraction on the flow of water through interstices of such an infinite variety as is afforded by the range of materials from fine clay to large boulders, such "hair-splitting" refinements of calculation would have very little practical value for this purpose, on account of the uncertainties which attend the determination of the variations of actual conditions to be met.

Causes of Failure of Sheet-Pile Dams.—Failures in sheet-pile dams have generally occurred in those of the spillway or overflow type, which are commonly called diversion weirs. The seepage water which finds its way under a line of sheet-piling tends to buoy up the particles of sand on the down-stream side, while the surface of the river bed is frequently subjected to a variable degree of scouring action from the current of water which spills over the crest. These two forces combined move the finer particles of sand first, then, as the interstices in the sand gradually increase in size, the upward flow of seepage water increases, both in force and volume, and the destruction of the dam is in progress.

It is customary with many engineers to make the length of sheet-piling such that the depth of penetration below the bed of the stream will be approximately equal to the head of water to be impounded by the dam or weir, and it is likely that many of the failures of such structures, which have been attributed to "unknown" causes, were due to the inadequacy of the cut-off wall under the structure—more often, however, on account of carelessness in driving and faulty alignment than from insufficient depth of penetration.

Length of Sheet-Piling Required.—Capillary attraction and surface friction, in the interstices between the grains, are forces of such magnitude in retarding the velocity of seepage flow through sand, or other pervious materials, that, with reasonable depth of penetration of a curtain-wall of sheet-piling, the direct pressure which may result from any reasonable head of water impounded by the dam cannot produce a velocity of seepage flow sufficiently rapid to move the fine particles of sand without the added force of the overflow current flowing on the surface of the river bed; therefore, in designing the dam or diversion weir, adequate provision must be made to counteract the effects of both of these forces.

Assuming that the force of capillary attraction effectively resists the pressure head of the impounded water, in its natural function of

producing an acceleration of seepage flow, then the body of sand on the down-stream side may be considered as a mass which is held together by the force of capillary attraction, and that the pressure due to the head of water on the up-stream side passes under the line of sheet-piling and acts on a wedge-shaped mass of sand on the down-stream side.

The hydrostatic pressure at the bottom of the sheet-piling is exerted equally in all directions, but, we are here concerned only with the force which acts in an upward direction. The tendency of this force to buoy up the mass of sand is resisted by the excess weight, or specific gravity, of the solid particles which compose the body of sand, after deducting the percentage of voids which are taken up by the water.

Let h = Maximum head of water on up-stream side,
d = Depth of penetration for sheet-piling,
s = Specific gravity of material penetrated,
x = Proportion of solids.

For average sand, $s = 2.65$ and $x = 1 - 0.40 = 0.6$.

Then $d = \frac{h}{s \times x} = \frac{h}{2.65 \times 0.6} = \frac{h}{1.59} = 0.629\ h$, which is the theoretic penetration.

This equation provides for the depth of penetration at which the excess hydrostatic pressure on the up-stream side is theoretically counterbalanced by the specific gravity of the depth of material on the down-stream side, in sand of such size and porosity that the combined effect of friction and capillary attraction is a maximum.

A length of sheet-piling which provides for a depth of penetration equal to the maximum head at flood-stage, therefore, involves a factor of safety of only 1.59 (with a tight wall of sheet-piling in perfect alignment) applied to the theoretic minimum. This is manifestly inadequate for such variable and uncertain conditions as are likely to be encountered.

Where the workmanship, alignment, and depth of penetration of a curtain-wall of sheet-piling are such as to ensure that there will be no marked "leakage" flow through openings, or places of variable density, silt and matter carried in suspension by the current of the stream will settle in the quiet pool of water impounded by the dam, and gradually seal up the interstices in the sand, until the bed of the stream becomes practically impervious. This has been thoroughly demonstrated in

irrigation canals constructed in sandy soil, and by the fact that the graded sands used in the filter beds of many of our larger cities require frequent renewal, or removal, for the purpose of washing out such accumulations, in order that even the customary slow rate of percolation may be maintained.

No material, other than solid bed-rock, offers a more secure support than sand which is properly confined; therefore, to make it serve as a safe foundation for a permanent dam is entirely a problem of making adequate provision against the only disturbing element—a current of flowing water.

For dams which are to be constructed on a sand foundation, and embody the special features of construction recommended in a subsequent part of this paper, the writer suggests the following empirical rules:

$d = 2\ h$.....For low heads, up to 8 ft.....(I)

$d = h + 5$.....For heads up to 25 ft.....(II)

$d = h$.....For heads of more than 30 ft.....(III)

$d = 0.66\ h$.....Minimum for any case.....(IV)

Special Features of Construction.—The horizontal distance through which seepage water must flow, after passing under a line of sheet-piling, before it reaches an outlet at the surface, is an important element in retarding its force to such extent that there will be no erosion, or movement, of the sand composing the surface of the river bed. It is especially important below spillways, or dams of the overfall type, where the velocity of current from the overflow of the dam may already be such that erosion of the finer particles of sand takes place; therefore, the added impetus of upward percolation would produce a dangerous condition of erosion and the inevitable destruction of the dam.

An apron below a dam serves, not only to resist the shock or abrasion of falling water, but also to seal the bed of the stream, so that the seepage water which finds its way under the curtain-wall of sheet-piling, must flow diagonally upward beyond the limits of the apron before issuing. The width of such apron, therefore, is an important element in fixing the practical factor of safety of the structure, and should not be overlooked.

Aprons for spillways, diversion weirs, and dams of the overfall type, should be submerged, so as to form a water-cushion to absorb the

shocks of falling water, ice, and débris. Such water-cushion should have a depth equal to one-quarter, and preferably one-third, of the total height of overfall, and a down-stream width of not less than 1½ times the total height of overfall, with the outer edge forming an apron, sloping upward, and extending down stream for a distance of not less than 1½ to 2 times the length of sheet-piling used in the main curtain-wall, ending with, and supported on, a secondary line of sheet-piling of from one-third to one-half the depth of penetration of the curtain-wall.

A special feature of construction, which provides for considerable economy in the quantity of concrete required for a wide apron, and, at the same time, provides a greater element of safety against the force of seepage flow than a solid concrete apron of the dimensions stated above, is illustrated in Fig. 1.

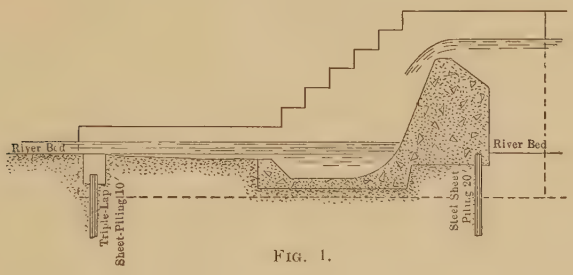


FIG. 1.

The submerged apron should be constructed in the form of a trough, with a depth of not less than one-fourth, and a down-stream width of 1½ times the height of the overfall, and the outer edge should be finished off at, or slightly below, the elevation of the bed of the stream. At a point down stream, a distance of about twice the length of the main sheet-piling used (or 4 or 5 times the head), there should be driven a second cut-off wall of tight sheet-piling with a depth of penetration of about one-half that of the main curtain-wall, parallel to the line of the dam, to support a substantial concrete curb which should be finished off at the elevation of the bed of the stream.

The writer wishes to emphasize the statement that: One of the most serious problems involved in the design and construction of dams for rivers with beds of shifting sands is that of providing adequate protection against dangerous erosion of the river bed for

some distance below the dam, where the erosive action of the current is dangerously magnified by reason of the upward force of seepage flow, under the variable hydrostatic pressure communicated from the head of water impounded on the upper side of the dam.

In the design suggested in Fig. 1, the open space between the edge of the trough forming the water-cushion and the concrete curb supported by the secondary line of sheet-piling provides an open space for the escape of the seepage flow which finds its way under the curtain-wall, while the concrete curb prevents the erosion and removal of the sand, which, in this confined section of the river bed, is subject to the combined forces of upward and horizontal currents.

The approximate general dimensions suggested apply to low dams of the diversion-weir class, such as are used to divert water from wide and shallow streams for purposes of irrigation and power; and the importance of careful and conscientious workmanship on every detail of construction in sand is so self-evident that special mention seems to be unnecessary.

Every dam or diversion weir should be designed in accordance with the practical demands of the exact local conditions which prevail. This involves modifications of any set of general rules, especially for higher heads, that is, for dams which, in sandy rivers, will raise the level of the water more than 12 or 15 ft. For low dams there is a variety of shapes, or cross-sections, from which to choose; but, for higher heads, economy in the matter of the materials involved narrows down the choice in the direction of the curved or ogee form of cross-section.

Under the general rules previously stated, a dam which is designed to raise the level of the water 50 ft., in a river having a bed of sand of unlimited depth, would require a curtain-wall of sheet-piling having a depth of penetration of not less than 80 ft. below the bed of the stream, as the minimum. A dam of this height would probably be designed with an ogee form of curved cross-section. The impracticability of driving a line of wooden sheet-piling to such depth in sand and gravel, and the expense of providing a curtain-wall of steel sheet-piling of such lengths, would suggest a careful analysis of modifications, for the possibility of eliminating a portion of such construction without impairing its stability and permanence.

Ample assurance of safety could be attained by using a line of

steel sheet-piling of such length as to provide a depth of penetration of 50 ft. under the up-stream side of the dam, and a secondary line of sheet-piling with a depth of penetration of about 15 ft. under the extreme toe, which should be curved up so as to form a trough at a depth of about 5 ft. below the bed of the river, for a water-cushion. Then, at a point about 200 or 250 ft. down stream, construct a smaller dam, about 15 or 16 ft. in height, with a line of main sheet-piling having a depth of penetration of 30 ft., and embodying all other features of design in accordance with the special features outlined in this paper, to back up the water against the main dam for a proper depth of water-cushion. The seepage flow, which would probably find its way under the main dam in considerable quantities, would issue from the section of confined river bed between the two dams, where the depth and length of the water-cushion thus provided prevents the removal of any sand, because, even during periods of storm flow, the violent abrasive force of the flood passing over the dam would be effectually broken by the extra depth of water-cushion which is provided by the concrete trough at the toe of the dam.

In calculating the proportions of a dam which is to be founded on sand, there is involved the consideration of the hydrostatic pressure against the under side of the dam.

The calculations for the cross-section of a masonry dam founded on bed-rock are based on the assumption of an absolutely water-tight contact between the footings and the bed-rock, therefore the entire cross-sectional weight of the dam may be used in calculating the moment of resistance to overturning.

In a dam of such proportions that the resultant of forces passes close to the toe, the effect of water working its way under the footings at points of defective contact, or through defective joints in the lower courses of masonry, might produce a change in the direction of the resultant such that it would pass outside of the toe. That section of the dam would then be in unstable equilibrium, and, unless restrained by other forces, would overturn.

In considering the forces which may act on a dam resting on sand, saturation of the foundation, with upward hydrostatic pressure, is an accepted fact, and, therefore, must be considered as a definite element in the calculations.

As stated previously, the combined effect of friction and capillary attraction offers such marked resistance to the flow of water through

sand that the velocity which results from any head, or inclination of surface, is infinitesimal, in comparison with the velocity which would result under similar conditions of head in a body of free water flowing in an open channel, or through a pipe. The transmission of hydrostatic pressure through a similar medium is similarly resisted, to a slight degree, as will be shown.

The removal of the soil above a stratum of dense clay, which in turn overlies a stratum of water-bearing sand or gravel under Artesian pressure, will disclose a moist degree of saturation of the clay such that water will accumulate in small pockets, or sumps, which may be bored or dug to shallow depths of 1 or 2 ft. into the clay. The seepage flow which is thus indicated is absorbed by the soil above by capillary attraction, and is evaporated from the surface; however, no hydrostatic pressure, of measurable magnitude, would be exerted against the footing of any structure resting on such a stratum of clay.

Hydrostatic pressure is directly proportional to the head of water, and time is not a factor in determining its magnitude, hence the coefficient of friction becomes zero. Clay contains a relatively large proportion of voids of infinitesimal size, and, while a high head of water pressing against the under side of a stratum of clay which is confined between two porous strata would undoubtedly transmit hydrostatic pressure of measurable magnitude through such stratum of clay, it is evident that, under moderate heads, the force of capillary attraction (in interstices of such infinitesimal size that they would almost warrant an assumption of molecular friction), is of sufficient magnitude to counteract and practically nullify the effect of hydrostatic pressure.

It may be concluded, therefore, that the force with which hydrostatic pressure is transmitted through alluvial soils is modified by a coefficient which varies as the square, cube, or perhaps "nth" power of the effective size of the interstices, or, inversely as such function of the effective size of the grains.

For the practical purposes of this paper, the probable existence of such a coefficient may be disregarded, because, for the interstices of the sizes which are ordinarily found in sand of 40% porosity, its probable value becomes so small as to be negligible.

The hydrostatic pressure against the under side of any dam of the

type and general dimensions illustrated in Figs. 1 and 2 is governed by the head of water on the down-stream side of the dam, because the pressure which is transmitted under the curtain-wall of sheet-piling, from the head of water above the dam, is afforded lateral outlet, therefore its intensity is limited to the depth of resistance, or head, opposing such outlet on the down-stream side.

The hydrostatic uplift per square foot of surface against the base of a dam of the type and general proportions recommended in this paper, therefore, will vary between 25 and 40% of that which would result from the head of water on the up-stream side.

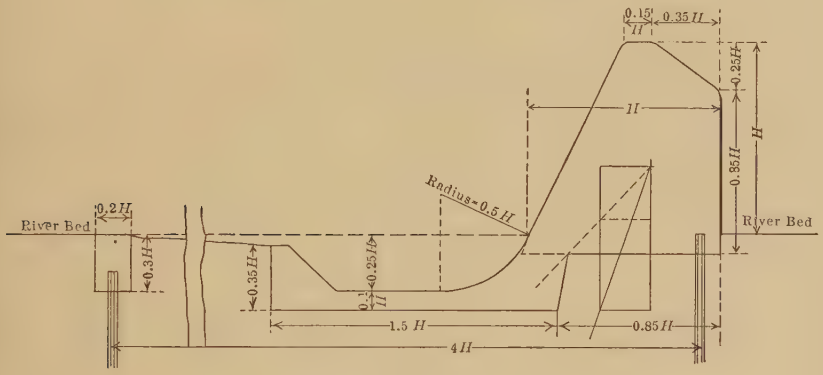


FIG. 2.

Fig. 2 illustrates the general proportions of the spillway section of a dam of the type discussed herein, with all dimensions given in terms of H, the height of the crest above the bed of the river, and assuming a length of spillway such that the maximum height of overflow will not exceed $\frac{1}{5}$ H.

The resultant, which is obtained when the calculated weight of the section of concrete is reduced by the total amount of hydrostatic uplift due to a down-stream head $= 0.3$ H, intersects the base practically in the center, as is shown by the solid line. This would indicate that the design may be modified so as to involve less concrete, but, the quantity to be thus saved, in the total cost of a low dam, will not outweigh the desirability of such a factor of stability in a structure of this class.

The dotted line indicates the resultant which would be obtained if the weight of the concrete were reduced by an amount equal to the full hydrostatic uplift which would result from a head equal to the maximum up-stream head $= 1.3\ H$. This resultant intersects the base outside of the middle third, but well within the toe.

The sloping portion of the top of the dam, on the up-stream side of the crest, is desirable to facilitate the passage of ice and floating débris over the crest, and to reduce the moment of ice thrust during periods of extreme cold when the entire flow of the stream may be diverted for power purposes.

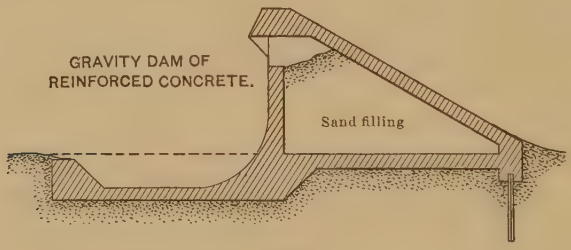


FIG. 3.

Any increase in the width of the base, with the additional material on the up-stream side of the dam, so as to lengthen the inclined portion, affects the moment of stability of the structure, on the principle of the well-known "gravity" type of dam.

Dams to be constructed on alluvial foundations should be designed with the maximum width of base which is consistent with economy; therefore, the gravity type of dam, constructed of reinforced concrete, is especially adapted to the requirements.

In low wooden structures of the gravity type, there is danger that they may float during a period of storm-flow of such depth that the gravity head, on which such dams depend for their stability, may be destroyed. By the use of reinforced concrete, however, this source of danger may be eliminated.

Figs. 3 and 4 indicate two variations of design for gravity dams of reinforced concrete which permit of removing the forms and filling the compartments with sand or other heavy material, in order to give additional weight to the structures, or, if suitable air vents

are provided, the compartments may be allowed to become filled with water.

From the simple inclined platform to the large hollow structures of the Ambursen type, the gravity dam of reinforced concrete offers

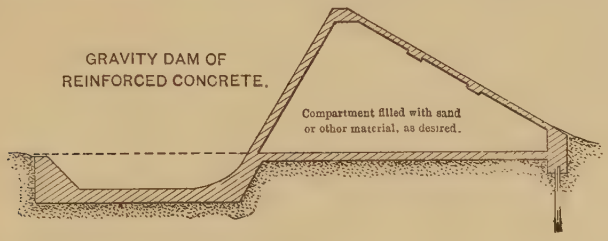


FIG. 4.

a sufficiently wide range of variation to meet the requirements of almost any locality, or any individuality of taste in engineering design.

DISCUSSION

Mr. Price. W. G. Price, M. Am. Soc. C. E. (by letter).—The writer has learned by experience that it is possible to construct on a sand foundation a dam which cannot be carried away by the force of flood-waters, although the cost will usually be greater than when there is a rock foundation.

The design must be such that parts of the dam cannot be separated from the main part by any force of water which will come against it. The whole structure must be heavy, and so broad that it cannot be moved bodily down stream, or be overturned. It must be very flexible throughout, so that when the water cuts the sand or gravel out from under it, the dam will bend and stretch, and sink into the cavity without breaking, and the material which goes into the cavity must then be replaced with the same kind of material on top of the dam; and this must be repeated until the force of the water can dig no deeper.

If the banks of the stream are also composed of material which can be eroded, the structure must be carried far enough back and over the bank, and made of sufficient vertical thickness to enable it to drop flexibly and fill any cavity which may be cut out under it.

There is a limit to the depth to which the force of the water can undermine and sink a structure of this kind, and, when that limit has been reached, the top part of the dam, or that part which is exposed to the air, can be finished in concrete.

Sheet-piling may be necessary; but there must be no piling under or attached to the dam, which will prevent it from sinking.

The writer has built a number of dams on this plan, where the sand was too deep for a rock or other secure foundation to be reached, and they have not been carried away. He constructed the two dams in the Atchafalaya River at Simmesport, La., the type being conceived and designed by him, as above described. They were completed in 1890, and are still in good condition.

Under these dams there was a stratum of fine sand 80 ft. thick, and it extended for miles on each side of the river. The depth of the river at low water was about 50 ft., and the dams in the center were depressed to 5 ft. below low water. During floods the river rises 50 ft. above the dams, and the velocity of the water directly over them is then more than 20 ft. per sec., and the discharge nearly 500 000 cu. ft. per sec.

It is the writer's opinion that permanent dams on this plan can be constructed across any alluvial stream, such as the Missouri, Mississippi, or the Colorado, with its bed of very fine silt. The water may dig 50 ft. or more in depth before it gives up the fight to destroy the dam.

ALEXANDER POTTER, ASSOC. M. AM. SOC. C. E. (by letter).—By the census recently published, many cities in the Western States show increases of from 300 to 500% in the last decade, and in many cases there is nothing to prevent their continued growth. This condition of abnormal growth also exists in the Canadian Northwest. To meet the increasing municipal and commercial needs of these cities, it is becoming necessary to harness the rivers and streams, but on many of them it is impossible, and on others impracticable, for economic reasons, to found the necessary structures on rock. This paper, therefore, is timely, and it is to be hoped that those who have had experience in founding large structures of this type on what are considered to be unstable foundations, will relate the results of their experience. Mr. Potter.

On many of these Western rivers it becomes necessary to construct dams which do not extend across the full width of the permanent valley. The alluvial deposits often form a natural dam for the greater part of the total width of the original valley, and the rate of percolation through this natural dam becomes an important feature in the economy of the design. In such cases the determination of the rate of percolation, therefore, is also important.

It will be conceded that the main problem is to prevent even the slightest movement in the material on which the dam is founded, and it is pleasing to note that the author's experience with quicksand as a foundation warrants his conclusion that it has lost the terrors which it ordinarily arouses.

The writer was recently forced to face the problem of designing a dam under just such conditions as are referred to by the author, and he was impressed by the lack of precedent and information concerning such an important branch of engineering. Therefore, he voices the hope of the author that this paper will bring out a full and free discussion from those who have had experience in the construction of dams which have not been founded on solid material.

For the depth of penetration of sheet-piling the author suggests a general rule which presupposes the ability to procure a continuous line of water-tight sheet-piling to the depth he mentions. For high dams, wooden sheet-piling is out of the question, because of the inability to drive it when the dam is founded on material requiring a great depth of penetration. Even with dams of less height, the difficulty of securing a continuous line of tight sheet-piling is great, and the cases on record where a continuous and tight sheet-piling has been driven to a depth of 30 ft. or more, are few.

The use of steel sheet-piling for a cut-off wall of a reservoir may be open to criticism because of the possibility and probability of abrasion of the protective coating on the steel in the process of driving. The extent of the deterioration of the steel when used for such

Mr. Potter.

construction is of great importance, and the writer has found it difficult to secure any reliable evidence as to the lasting qualities of steel when used under somewhat similar conditions. Contributions from engineers having experience with steel sheet-piling, under conditions which could be taken as precedent for a steel cut-off wall, should form a very valuable feature of the discussion of this paper.

The writer's attention has recently been called to a process of constructing a continuous concrete cut-off wall of any depth by first driving square timbers to which are fastened steel channels. The timbers between the channels are removed one at a time, the channels being left in place for forms, and the space between them filled with concrete. When the concrete has set, the steel channels are removed. The writer trusts that some one familiar with this method will describe it.

The author's remarks on the percolation of water through sand and their application to show one of the principal causes of failure of sheet-piling dams are very interesting. Attention might be called to another cause of failure of such structures not founded on rock, and one which has not received the study it merits. In a structure of this type of any considerable magnitude, it is impossible to prevent unequal settlement. Such settlement is due principally to the non-homogeneity of the compressible foundation; sometimes, in a poorly designed structure, it is due to the excessive foundation pressures developed at various points. Provision should be made for unequal settlement in such a structure, as it is often sufficient to rupture it completely at one or more points. These settlement cracks, often of considerable magnitude, are very serious when they develop in the lower part of the dam. A small stream of water begins to percolate through the crack, and its tendency is to undermine the foundation gradually by removing the fine material. This undermining and settlement once begun, is progressive, and the ultimate failure of the structure is only a question of time.

Settlement joints—or, as they are sometimes erroneously called, "expansion joints"—should extend entirely through the structure, and permit of a liberal movement in any direction at the joint. There appears to be no reason why these joints cannot be made permanently water-tight with either lead or copper plates. The distance between such joints is at present an open question, but it should not exceed 200 ft.

Unequal settlement and percolation under a dam can be prevented to a very great extent by increasing the width of the base, within reasonable limits of economy. The width of the base adds to the stability far more than any other single factor.

Attention is also called to the fact that the author has not brought out with sufficient force the necessity of relieving, by a system of drains, the hydrostatic pressure constantly developing under the floor

of the dam. The necessity of relieving this pressure as much as possible is as great, for the comparatively narrow, solid, gravity section, as for the hollow, reinforced concrete dam, if not greater. Mr. Potter.

T. Kennard Thomson, M. Am. Soc. C. E.—If a rock foundation cannot be obtained, either because there is no rock or on account of inability to raise funds for a permanent dam, a dam on sand foundations may be quite justified, provided it is not made too high. There is no doubt that sheet-piling would greatly retard the flow of water under a dam, but it would be very dangerous to assume that the results of this retarding could be determined definitely by calculation, for, as the author very properly says, the failures are more often "on account of carelessness in driving and faulty alignment than from insufficient depth of penetration." This carelessness or unintentionally poor work, however, is always likely to occur. Cases are well known where piles, driven with considerable care, have afterward been found to have been telescoped or badly bushed, without having gone down to the proper depth or as deep as supposed. Many have seen wooden sheet-piling, driven with the utmost care, with tongued and grooved or similar joints, which appeared to be in perfect alignment on the surface, and yet the piles have been forced apart below ground. In the Brooklyn Navy Yard some steel sheet-piling had been, apparently, perfectly driven, the tops showing in true alignment, but the bottoms were twisted, bent, and coiled in all imaginable shapes; and yet the piling consisted of 8- and 10-in. channels separated by 6-in. timbers and firmly bolted together. It would be difficult to get a stronger sheet-piling. Mr. Thomson.

For these reasons, the speaker would not like to rely on driving sheet-piling to such a depth as 50 ft., as suggested by the author; for, if the conditions were such that one could be sure of driving it to such depths, the ground would be so soft as to be unsafe. As it is impossible to ascertain the condition of driven piling, one must take a good deal for granted. The speaker does not, by any means, wish to infer that he would not recommend sheet-piling to retard the flow; he simply wishes to utter a warning against placing too much reliance on such protection.

The sections, shown by the author—Figs. 1, 2, 3, and 4—for concrete dams on a sand or earth foundation, would be sure to develop bad cracks. In fact, the speaker does not consider such masonry dams at all appropriate for a sand or loam foundation, as the foundation is more or less yielding, while the structure above is naturally unyielding, until collapse. It would seem that an earth dam with a core-wall would be much more appropriate. The sheeting could be driven under or near the core-wall.

G. E. P. Smith, Assoc. M. Am. Soc. C. E. (by letter).—Dams on sand foundations are now quite generally thought to be as necessary Mr. Smith.

Mr. Smith.

engineering projects as those on rock bottoms. The examples of such structures now in successful use show wide variation and also great ingenuity in their design. There have been failures of such dams, usually due to unprecedented floods for which the dams were not designed, or to the lack of engineering direction in their design or in their construction, or to the slow deterioration of some portion of the structures without attention to their up-keep. Any of these causes may be equally disastrous to dams on bed-rock.

As a preliminary to the discussion of dams on sand beds, the laws of flow through saturated pervious deposits should be understood. The following remarks have reference to those cases in which it is impracticable to shut off the underflow of water by a curtain-wall extending to an impervious stratum.

The resistance to seepage flow is that due to water friction, both internal and surface, and depends therefore on the character of the medium and, to a lesser extent, on the viscosity of the water. Capillary attraction does not exist as a force in saturated deposits. It has been shown by investigators that the velocity of seepage flow is a function of the porosity of the material, of the "effective size" of grains, of the hydraulic gradient, and of the temperature, each in an important degree.

The "effective size" of a sand was defined by Hazen as that size, than which 90% by weight of the material is coarser and 10% is finer, a definition based on experiments with Massachusetts deposits. The resulting formula coefficients, however, are not of wide application, and have been shown to be much in error when applied to river sands in the Rocky Mountains. Grading sands with King's aspirator is a more rational method, but in this case, also, the coefficient in the flow formula and the graduation table are dependent on the sand used in the rating. No satisfactory basis has yet been found for classifying sands and gravels. The effects of the sharpness or roundness of grain, of the various crystalline constituents (for example, mica), and especially of the peculiar stratification of sand *in situ* due to its sorting by running water, seem quite incapable of mathematical expression. Therefore the formulas for seepage flow are useful for deducing comparative results in a region, rather than for obtaining absolute values. A recent article in the technical press, describing the construction of weirs on sand foundations in India, proposes empirical formulas for general use in designing such structures. It is possible that the formulas should be much modified for use in America, or rather, in each section of America.

It is to be noted that the velocity of true seepage flow varies approximately as the first power of the pressure head, or of the gradient, and hence the ordinary formula for free flow,

$$v = \sqrt{2\,g\,h},$$

is inapplicable to this discussion. With very coarse sorted materials, the flow is not exactly seepage, but is partly free flow. Mr. Smith.

The flow through river sands *in situ* is of greater interest than that of flow through screened media. Much experimental knowledge of the former has been obtained by Professor C. S. Slichter, who devised an electrolyte method of measuring the rate of movement of ground-water, and has carried on investigations in many localities, from Long Island to California. His results are available in the Water Supply Papers of the U. S. Geological Survey. The average velocity of underflow in the Arkansas River sand was found by Slichter to be 7.4 ft. per 24 hours, while the maximum value obtained was 22.9 ft. The latter value is equivalent to a velocity of about $\frac{1}{8}$ in. per sec. with a hydraulic gradient of 1:10. The writer found maximum velocities of underflow of more than 400 ft. per day on an Arizona stream with a gradient of 20 ft. per mile. Before designing a dam or weir on a sand foundation, where the perviousness of the sand is unknown, this factor should be determined by direct measurements.

The effects of seepage flow beneath a structure are of two kinds: first, the grains of the material may be moved, particularly at the toe of the apron, where a point of maximum seepage velocity exists; second, the distribution and amount of upward pressure on the structure is modified by the seepage, and may be the primary cause of failure, especially if there is a wide apron. The first effect depends on the velocity of the seepage and the size of the grains of sand, and may be made inactive by lengthening the path of the overflow. The danger is reduced by placing a rip-rap of boulders at the toe, the smallest ones at the bottom, the largest ones on top. The second effect is dependent on the design of the foundation and base of the structure.

In his treatment of sheet-piling, the author has endeavored to establish a relationship between the depth of penetration and the head of water behind the dam. It would seem, however, that the piling should not be considered as an independent feature of the design. The same result can be attained by a long base with even no piling at all, or by a short base with piling, and the choice of the final design is largely an economic one. In the Laguna Dam very short piling was used, and its effect in "discouraging" the underflow of water is not great. In that case rock filling was obtained cheaply, and the wide base was adopted.

To illustrate the effect of a curtain-wall on the stability of a structure, and to show the general dependence of the problem on the law of seepage flow, the following approximate analysis is made. In Fig. 5 the ratios of base, head, and depth of penetration are retained as in Fig. 2, but the figure is simplified for the purpose of analysis.

Mr. Smith.

Let H = maximum head of water between heel and toe of dam, usually occurring at low water;
d ($= -1.6H$) = depth of penetration;
L ($= 1.5d$) = length of base;
h' = head lost on the right of the curtain-wall;
h'' = " " " " left " " " "
C = a constant, depending on the character of the sand;
Q = quantity of seepage flow per unit of time.

Let ABC be a differential element of flow. The assumptions as to the length of the element and its triangular shape are approximate, and introduce errors, which, however, tend to balance each other. The sand is assumed to be uniform in character.

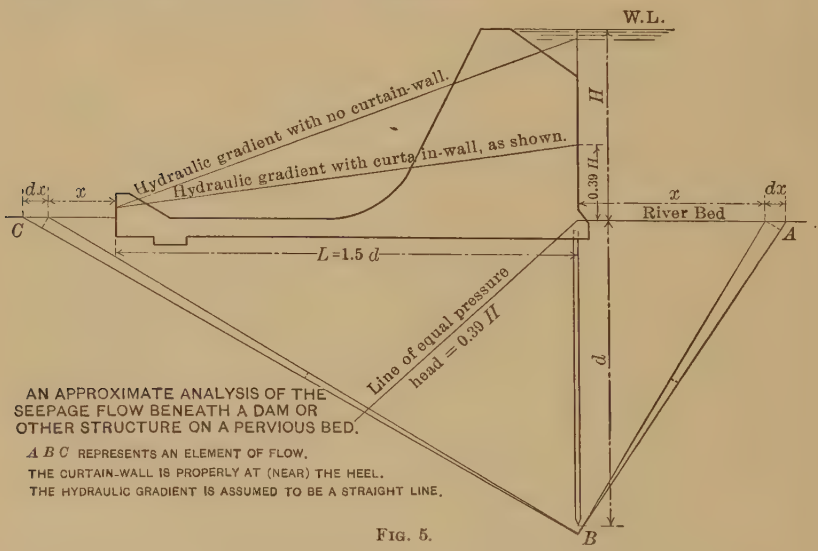


FIG. 5.

The formula for seepage flow may be written thus:

$$Q = C\frac{H\,S}{L},$$

in which S is the cross-sectional area. In the differential triangle, on the up-stream side, the average cross-section is

$$S = \frac{1}{2}\frac{d}{\sqrt{d^2 + x^2}}\,dx.$$

Then

$$dQ = \frac{Ch'}{2}\frac{d}{d^2 + x^2}\,dx;$$

$$Q = \left[\frac{Ch'}{2}\tan.^{-1}\frac{x}{d}\right]_o^\infty = \frac{1}{4}\pi Ch';$$

$$h' = 1.27\,\frac{Q}{C}.$$

Mr. Smith.

Similarly, on the down-stream side

$$S = \frac{3}{4} \frac{d}{\sqrt{x^2 + 3dx + 3.25d^2}} dx.$$

Whence

$$h'' = 2.3 \frac{Q}{C}.$$

Therefore, about one-third of the head, H, is lost, or neutralized, on the up-stream side of the piling. The upward pressure at the heel of the dam is not measured by the pressure at the point of the piling. There is a further loss of head in the flow upward, and evidently it may be taken at two-fifths of h'', so that the upward pressure at the heel is equal to three-fifths of h'', and hence equals $0.39H$. This heel pressure is shown in Fig. 5 as the ordinate used to determine the hydraulic gradient. The upward pressure decreases from the heel to a small value at the toe. The gradient is subject to a correction dependent on the depth of the base below the river-bed.

As stated above, this demonstration is illustrative merely, but it exhibits the principle that the problem involves all the lines of flow. To consider only that line of flow which follows along the piling and the base of the dam is erroneous.

Without the curtain-wall, the hydraulic gradient would be the higher line shown in Fig. 5, allowance being made for losses at entrance and exit. Therefore the effect of the curtain-wall, for the given ratio of base to depth of penetration, is to reduce the upward pressure on the structure by about one-half. The velocity of flow will also be diminished about one-half by the curtain-wall, inasmuch as the minimum length of the flow line is increased from $1.5d$ to $2.8d$.

The effect of another line of sheet-piling placed near the toe of the structure would be to reduce the velocity of the underflow, but to increase the upward pressure. Sheet-piling is most effective when concentrated at or near the heel, and piling which is water-tight should not be used under the toe or beneath the apron. Referring to the detached line of piling and concrete curb, as proposed by the author and shown in Figs. 1 and 2, would not the effect be to increase the upward seepage flow between the curb and the toe of the dam, and thus be an element of weakness rather than safety? In some of the rivers of the West—the Gila, for example—detached piling of that character is apt to disappear during high floods.

At the writer's suggestion, Professor H. C. Wolff, of the mathematics department, University of Wisconsin, has undertaken the rigorous solution of the problem of underflow beneath a dam. It involves the use of a pressure function which is determined to satisfy Laplace's equation, with the imposed boundary conditions. The case is entirely analogous to problems of the flow of heat.

The results of Professor Wolff's analysis, applied to the structure

Mr. Smith.

discussed above, are shown in Fig. 6. The flow boundary is down the face of the piling, up the back, and along the base of the dam. The head is considered to be divided into ten equal parts, and the losses of the successive parts are indicated by the ten divisions along the boundary. The hydraulic, or pressure, gradient is then plotted in the lines, ab, $b'c$, and $c'd$. If H is assumed to be 10 ft., then each ordinate is shorter than the previous ordinate by 1 ft. The upward pressure on the dam varies from $0.36H$ at the heel to no pressure directly at the toe; but, at a short distance back from the toe, the pressure on the apron is considerable. The average upward pressure on the base is $0.27H$. Relatively high velocities of seepage flow occur at the point of the piling and at the toe of the dam, as indicated by the slope of the hydraulic gradient.

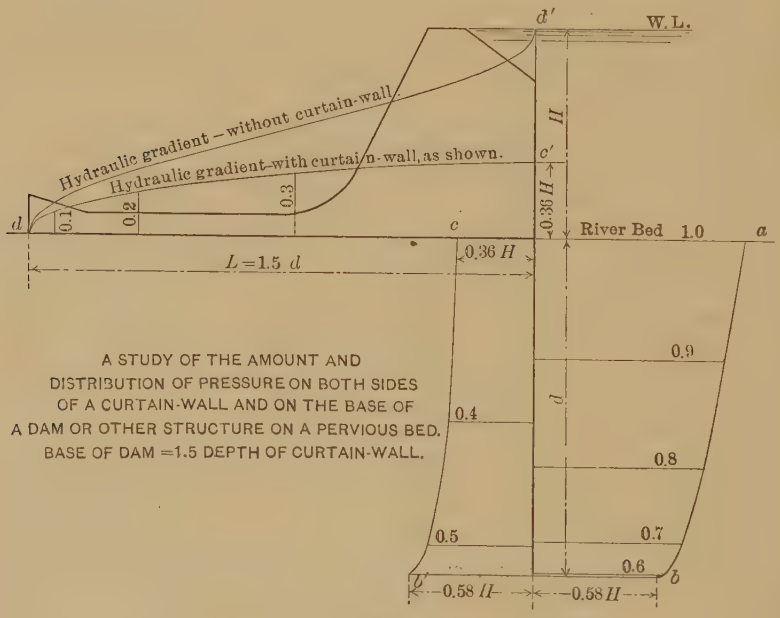


FIG. 6.

The line, $d'd$, represents the hydraulic gradient when there is no curtain-wall beneath the dam. The line is plotted from thirty-five computed points. It is a sine, or ogee, curve, and the maximum velocities occur at the heel and at the toe. The average upward pressure is $0.5H$. At one-tenth the base length from the toe the pressure is $0.21H$.

By applying the same method of analysis to a series of cases, in which the depth of penetration is taken at various percentages of the base, the curves of Fig. 7 are obtained. The curves show how the average upward pressure and the upward pressure at the heel of the

dam decrease as the depth of the piling increases. It is apparent that the benefit derived from the piling decreases with the depth—an example of the principle of diminishing returns. Mr. Smith.

It appears, also, that the uplift on the apron is much greater than it is ordinarily assumed. Aprons which are very thin in section are common. Much thought must be given to the design of the apron and of the overfall face.

The entire problem of dams and weirs on sandy river beds is one of unusual difficulty, and is complicated by uncertain factors such as

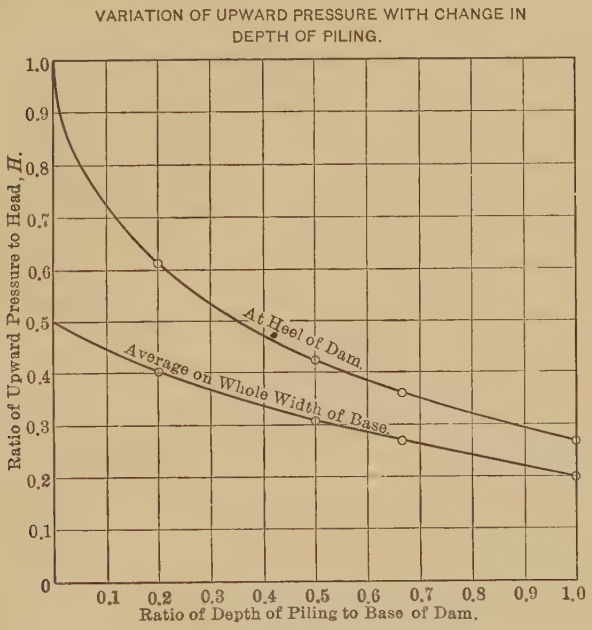


FIG. 7.

the effects of impact and cross-currents, and the extremely variable velocity of seepage in different sands. Doubtless our knowledge concerning such structures will be greatly increased in the future, as their number increases.

ALLEN HAZEN, M. AM. SOC. C. E. (by letter).—An investigation of the stability of dams built on sands and gravels is a most commendable undertaking, and is likely to lead to success in many cases. In reaching success, an adequate knowledge of the laws of the flow of water through sands and gravels is of fundamental importance, and the writer regrets to see such calculations discarded by the author. Mr. Hazen.

The results credited to the Massachusetts State Board of Health

Mr. Hazen.

were the work of the writer, about twenty years ago, at the Lawrence Experiment Station.*

These results have been quoted many times, but in doing this their form has been sometimes changed, new assumptions have been introduced, and limitations originally made have been omitted, so that, in using them at the present time, the only safe way is to refer to the original publication.

There is no reason why flows through gravels should follow the formula given for sand, as the author seems to suppose. In the original publication it was clearly stated that this was not the case, as appears from the following quotations:

"The frictional resistance of sand to water within certain limits of size of grain and rate of flow varies directly as the rate and as the depth of sand. This is given by Piefke as Darcy's law. I have found that the friction also varies with the temperature, being twice as great at the feezing point as at summer heat both for coarse and fine sands, and also that with different sands the resistance varies inversely as the square of the effective size of the sand grain."

The formula was then given for the flow of water in sands with effective sizes between 0.1 and 3.0 mm. It was then stated:

"For gravels with effective sizes above 3 millimeters the friction varies in such a way as to make the application of a general formula very difficult. As the size increases beyond this point, the velocity with a given head does not increase as rapidly as the square of the effective size; and with coarse gravels the velocity varies as the square root of the head instead of directly with the head as in sands. The influence of temperature also becomes less marked with the coarse gravels."

The table was therefore prepared showing the experimental results for gravels, not covered by the sand formula. It is also to be noted that in the original publication results were uniformly expressed in terms of meters daily in a solid column of water of the same area as that of the sand. In quoting the results in the present case the terms of the statement have been changed by assuming 40% of voids and by multiplying the figures by 2.5, besides otherwise changing the units of the formula.

The Lawrence formula may be relied on at this time, not alone or principally because of the relatively limited experience on which it was originally based, but because it has been widely applied since that time, and when properly applied has proved reliable.

It certainly does not discredit the formula that the coefficient varies with the character and cleanliness of the sand, and even to a certain extent with the "uniformity coefficient," which is a measure of the relative range in size of particles in the same sample. It is well known that there is a range in the coefficients to be used in computing the

* Described in the Annual Report for 1892, p. 539.

flow of water through pipes, but the fact that a higher coefficient is used for a clean and smooth pipe than for an old and dirty one is no reason for discarding the formula. In the flow of water through the much more complex passageways between the grains of sand there will obviously be variations in the coefficient, depending on the shape and smoothness of the particles, the way in which they are packed, and the quantity of finer foreign material also contained in those pores. As a matter of fact, the variation of c in the original formula, Mr. Hazen.

$$V = c\, d^2 \frac{h}{l} (.70 + .03\, t),$$

is not especially great. The coefficient rarely falls below 400, even for old and dirty sand, and rarely rises above 1 200, and, in a majority of ordinary sands, falls between 700 and 1 000. This is certainly not a wide range.

It is to be noted that the formula was never intended to apply to clays, hardpans, soils, and other materials. The effort to apply it to such materials is not to be encouraged, and the results are not to be depended on.

The procedure of sifting a sample of the sand through rated sieves, plotting the results, and determining the effective size and the uniformity coefficient, is a very definite one, and comparatively simple. Nevertheless, erroneous results have frequently been reached, by failure to observe the fundamental requirements.

It may not be amiss at this time to define the procedure as to some of the matters regarding which errors have been made most frequently.

The effective size is that size such that 10% by weight of the particles are smaller than it. The decision to use 10% was reached by a study of experimental results with various sands at the Lawrence Experiment Station. It was found that the 10% line was a distinctly better basis for arranging the experimental results than either the 8% line or the 12% line, and therefore it was selected. The data on which this decision was reached were good, although not numerous; subsequent experience, with a greater variety of sands, has not shown reason for reconsidering the decision first reached.

The diameter of a sand grain is defined as that of a sphere of equal volume. This must be rigorously insisted on. Taking the diameter of grains as equal to the measured mesh of sieve or some such arbitary basis invariably leads to errors which may be large.

The method of rating the sieves described in the original publication was simple and sufficient. It consisted in shaking sieves with a sample of representative material precisely as in the ordinary course of analysis, then taking the sieves apart, giving each sieve separately a slight further amount of shaking, when the particles passing are all of nearly the same size, which size is larger than nearly all the particles

Mr. Hazen.

that have passed and is smaller than nearly all the particles that remain on the sieve. Some of the particles resulting from this further shaking are weighed and counted, and the average diameter is calculated from the average weight; the size thus found is the size of separation of the sieve. This process is repeated several times, with representative sands of different types, and the average taken. The shaking is not to be continued until no more particles will pass, as often stated, but only until the bulk of that sand that will pass that sieve with a reasonable amount of shaking has passed, as, for example, to the point where doubling the number of shakes will not make considerable difference. There are always some holes larger than the average, and if shaking were continued until no more particles would pass, the process would be a very slow one, and the result would be to bring the size of separation up to the size of the largest holes.

The sieves must be of regularly woven wire cloth, and must be carefully rated. It has not been found possible to buy ready-made sieves which can be relied on without rating.

The process of mechanical sand analysis, as a basis for estimating the flow of water in sands and gravels, is emphatically not one that can be taken up in a spare hour, in a way to yield reliable results; but, a careful manipulator, by following a perfectly simple procedure, may reach results which are trustworthy, if not precise, and which, in the words of the original publication, enable him to:

"* * * decide with confidence many otherwise indefinite points, and thus avoid unnecessary expense and unsatisfactory results from the use of unsuitable or poorly arranged materials."

As an illustration of the practical use of this formula, take the case of the filter construction now under way at Toronto, Ont. A large permanent plant is being built on a sand island, cut by channels, with foundation some feet below lake level. It was proposed to dig a drainage canal entirely around the site and pump the water out of it, thereby draining the whole site. Calculation made in August, 1908, assumed this drainage canal to be 3 500 ft. long, and that the depth of material under the canal carrying water would be 50 ft., making a total area of 4 acres through which the water would flow. The sand was assumed to have an average effective size of 0.25 mm. and c was taken as 800. It was assumed that the average distance from the canal to the water outside would be 200 ft., and that the water in the canal would be 5 ft. below lake level, resulting in a ground-water slope to the canal of 0.025. At a temperature of 50° water would pass, by the formula, at the rate of 1.25 m. per day, or 1 110 000 Imp. gal. per acre daily, and for the 4 acres of assumed cross-section of flow the total quantity of water would be 4 440 000 Imp. gal. per day. The writer does not think that any of the practical men who looked at the site before it was drained believed that the removal of so small a

quantity of water would suffice. As a matter of fact, the actual quantity of water pumped has never exceeded the computed quantity, and has ordinarily been only one-half or one-third as much, owing in part to the fact that the sand proved to be finer than was assumed (actually about 0.21 mm. on an average), and that the average distance from the canal to open water was greater and the hydraulic slope less steep than assumed. Mr. Hazen.

The use of the formula in this case allowed the installation of suitable equipment at the start, avoided false moves, and allowed the structures to be built economically on a site which, in the absence of such calculation, might have been assumed to be unsuitable because of the difficulty of handling the water in the sand.

R. C. Beardsley, Esq. (by letter).—This subject is of the greatest interest to the writer, and though some of the argument seems to be faulty, the paper should be of value in direct proportion to the amount of discussion it brings out. Mr. Beardsley.

The writer has built some twenty dams on soft bottoms, about half of them being on sand. A number of the earlier dams met with accidents, and each failure taught a lesson. Mr. Koenig's statement that textbooks emphatically discourage the construction of dams on sand bottoms is quite inaccurate, as not a single textbook is recalled that does this. "Hydro-Electric Plants," by the writer, devotes considerable space to the subject. The high cost of such dams, when properly built, may amount to "emphatic discouragement" to many, for it is true that for sand bottoms the cost of foundations is much the largest portion of the total cost. The engineer is usually compelled to dispense with precautions which he may think quite necessary for the safety of the dam, in order to reduce the cost to some figure guessed at by the owner or obtained from a source unknown even to himself.

In the writer's experience, the failures above mentioned have in every case been due to inadequate aprons or extension mats. The ideal dam for sand bottoms is one consisting of three essential parts: (1) At the up-stream edge there must be a water-tight row of steel sheet-piling, driven to proper depth, and fastened to the edge of the dam in a water-tight manner. (2) The dam, proper, must be of permanent and water-tight materials, and of such section that the pressures are uniformly distributed over the entire base. (3) Extending along the down-stream side of the dam, some means must be provided to conduct the water which passes over the dam's crest in such a manner that no serious vibrations will be set up, no vacuums, and so that the undermining of the dam will be prevented.

Fig. 8 shows diagrammatically the ideal dam; the rectangle, *A B C D*, merely represents the uniformly distributed, effective weight acting on the sand bottom and tending to resist the tipping and sliding of the structure.

Mr. Beardsley.

The writer knows of only two types of dam which give the same pressure on every square foot of the foundation—the gravity dam with apron, and the reversed dam.

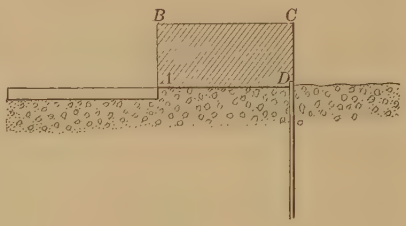


FIG. 8.

In Fig. 4 Mr. Koenig shows a gravity dam, and calls it an Ambursen dam. The gravity dam was patented by the writer's father, E. R. Beardsley, many years before the Ambursen Company went into the dam business. It was described by him in a pamphlet entitled "The Gravity Dam," published in 1895. Mr. Beardsley not only explained the entire theory of such dams, but gave to the dam its name. The reversed dam is a new type, lately invented by the writer.*

Either of these dams gives the desired distribution of pressure. All other dams have an excessive pressure at the down-stream edge, which, on sand bottoms, of course, is a disadvantage, such foundations having a questionable safe bearing value. Excessive pressures are especially to be avoided where there is the slightest danger of undermining.

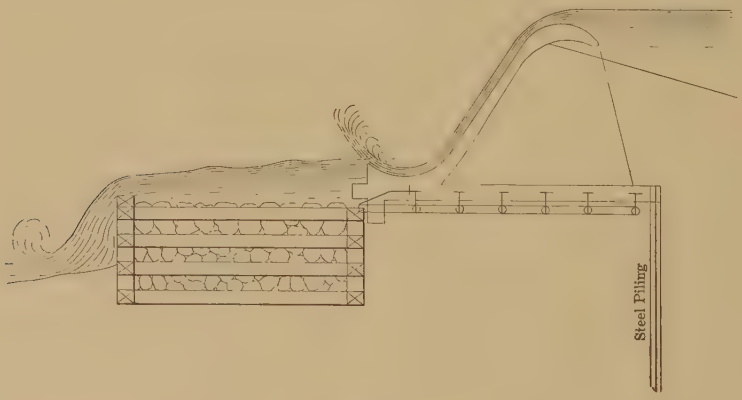


FIG. 9.

According to the writer's experience, an ideal dam for the most treacherous bottoms is the reversed dam, shown in Fig. 9. This uses the gravity principle, but, instead of carrying the water load on the deck, it carries a much larger (two to three times larger) water load on the base. The dam, as shown, is filled in above with sand, although for the usual soft bottom this would not be required. Such filling will

* Described in *Engineering Record*, November 26th, 1910; and *Engineering Contracting*, February 1st, 1911.

Mr. Beardsley.

give the maximum degree of safety, as regards tipping, sliding, and seepage. The apron of the dam is also provided with an unusual upward curve so as to aid in killing the velocity of the water leaving the dam. In the case of high dams, the lower portion of the apron would be covered with metal. The effect of such an apron should be similar to that of the deflector used for breaking up the discharge from high-head impulse wheels. The water falling from this curve would strike on the top of the cribbing, and escape, partly over the top and partly between the down-stream timbers, thus spreading the current and preventing the swirl which has proved fatal to most sand bottom dams. As the finest western sand will be eroded by a velocity of 1 ft. per sec., or less, and as it is a well-known fact that a swirl set up by water passing 4 ft. deep over a comparatively low dam will wash holes below the dam to a depth of 20 ft. or more, the importance of damping out all possible current will be appreciated. A crib mat is shown as an ideal. This type has many advantages, as it may settle without injury, is comparatively cheap, and has sufficient depth to distribute the escaping water. This mat must be anchored securely by chains or bolts, so that it cannot settle away from the dam.

In Mr. Koenig's paper too much importance has been given to up-lift under the dam, due to seepage. While every engineer should know that, unless such up-lift is eliminated, a failure is quite sure to follow, yet the elimination of such pressure is so easily and surely accomplished that it is not necessary to know how to calculate its intensity when allowed to exist. On the bottom of thousands of filter beds large quantities of water are collected through drains and conducted away from the beds without carrying away the sand. In the same way, the very small seepage under steel sheet-piling can be collected under the base of the dam and conducted safely to the water below.

The writer questions the existence of any considerable up-lift where only one row of piling is driven, and that along the up-stream edge, even though there is no resort to drainage; at least, he has never observed a pressure of more than 1 lb. per sq. in. under such conditions, even when wood piling was used. In all such river beds there is a constant underground flow, and part of the seepage under the piling will not come to the surface at all, but will continue on down the stream as ground-water. However, drainage costs so little that it should be installed in every case.

In the writer's opinion, the dam shown by Mr. Koenig in Fig. 1 is sure to fail unless the down-stream toe is so well reinforced that it cannot break. Two dams, built like that shown, failed. The pressures are very high at the toe—the point where the slope of the apron intersects the base—and, as indicated in Fig. 1, this point will become the tipping edge, unless reinforcement transfers the fulcrum to the down-stream edge of the mat. It would be necessary to have the top edge

Mr. Beardsley.

of the mat, at the down-stream edge, at a sufficient depth to reduce the velocity of the water escaping from the pocket to about 1 or 2 ft. per sec., otherwise the back-wash would undermine the mat (if as shown in that cut) in a short time.

Next to undermining below the dam, the greatest danger, the writer thinks, is that due to the horizontal down-stream thrust causing the entire dam to slide on the bed of sand.

Fig. 10 illustrates the very unstable condition of a dam resting on a bed of sand and with a deep hole washed out below it. In this case, the tendency is for the dam, and the entire mass of sand, *A B C D*, to slide down stream under the pressure represented by the pressure diagram, *D E F G*. The coefficient of friction along *A D* may be assumed as about 0.20 to 0.25, and the weight of materials must be calculated on the assumption that practically the entire structure and foundation is submerged and therefore has lost in weight 62.5 lb. per cu. ft.

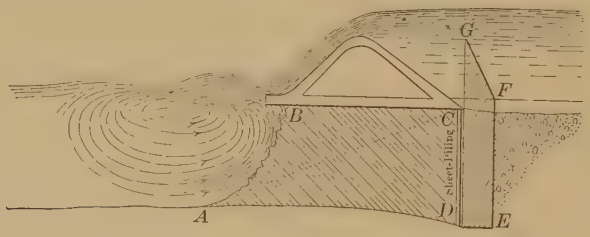


Fig. 10.

There is also the angle of repose of the sand, as *D A C*, which enters into the problem. This angle may be as small as 3° for gumbo, or from 10° or 15° for Western sand. It will also be noted that the water is anything but calm along the surface *A B*, and that, in many dams, the overfalling water sets up a rhythmic vibration similar to pressure waves in a long penstock, and thus greatly reduces the coefficient of friction and the angle of repose.

As to the general proportions: The writer has never known of the water breaking under a gravity dam mat from the up-stream side when a row of sheet-piling was driven along the up-stream edge. Of the three cases where the dam was completely washed out, it is positively known that two were undermined by the back-wash along the down-stream side, and the third was pushed out, as explained in connection with Fig. 10. The latter dam was on the Elkhorn River, in Nebraska, and the foundation consisted of the finest sand mixed with silt. In all cases the dams were provided with an extension mat, from 16 to 24 ft. long, and yet, in a number of instances, the back-scouring extended for a distance of from 40 to 60 ft. back under the dam. Had

Mr. Beardsley.

the extension mat been built so that it could settle, these accidents could not have happened. Piling should not be used for extension mats, and very seldom to support the dam, as such support takes the weight off the sand, and encourages a concentration of the seepage water. Wood sheet-piling should never be used unless the soil is free from large rock, and, in all cases, piles should be jetted down.

Crib mats of 10 by 10-in. timbers, filled with stone and sunk from 8 to 10 ft. in ordinary soil, cost about 15 cents per cu. ft., and, for a dam 20 ft. high, would cost from $50 to $60 per lin. ft. of dam, or considerably more than a complete 20-ft. dam would cost if built on rock bottom. To this cost must be added that of the sheet-piling. It has been the writer's custom to use piling at least 12 ft. long, and

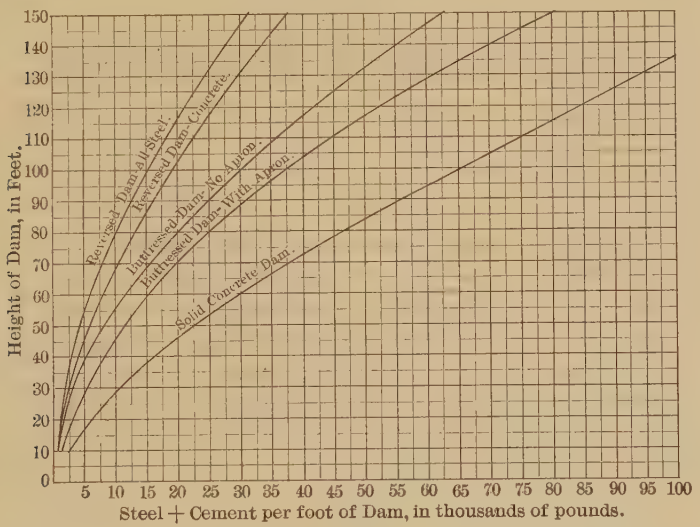


Fig. 11.

from that up to a length equal to the height of the dam. However, this practice has not been based on tests or actual knowledge of what the proper depth should be. By reference to Fig. 9, it will be noted that shorter piling would be required for a reversed dam than for any other, if the interior is filled with soil as indicated; for, in this case, a maximum length of seepage path to the top of the piling is secured without involving a flat surface, which might tend to cause the water to follow from the top surface of the fill down to the surface of the piling. Seepage will follow such surfaces for great distances, as in the case of pipes laid through earth dams, etc.

The greatest caution must be observed in keeping the foundation pressures well within safe limits. Thus, for sand bottoms mixed with

Mr. Beardsley.

more or less silt, Ira O. Baker, M. Am. Soc. C. E., states that the bearing value is practically zero. Such soil must be excavated, and a stratum must be reached which will support with safety the maximum pressures. A reversed dam 100 ft. high has a maximum foundation pressure of 5 000 lb. per sq. ft., or less, while the solid masonry dam, as usually built, has pressures exceeding 20 000 lb. per sq. ft.

Many failures are caused by inadequate foundations. Considerable time is required to make proper soundings, and the cost is a large item, hence, most dams are built by guess. The surface of the river bed is taken as a fair sample of the soil at greater depths, while, as a matter of fact, it is difficult to find a foundation which does not present much the best appearance on the surface.

The reversed dam is the cheapest of all permanent dams, and also the safest. Fig. 11 indicates the great saving in materials effected by it, and gives the relative weight of transported materials, steel + cement, for various types.

Mr. Koenig.

Arnold C. Koenig, Assoc. M. Am. Soc. C. E. (by letter).—From the discussion it would appear that some have not fully considered the importance and physical co-relation of the special features of the design illustrated by Figs. 1 and 2, which it was intended to emphasize as ensuring safe and permanent construction under the average conditions to be encountered in Western streams having "bottomless" beds of shifting sand.

Many of the criticisms will be found to be unsubstantial when compared with the corresponding subject-matter in the paper, but, both favorable and unfavorable criticisms are welcomed for their value in stimulating further consideration of the subject.

No single principle of engineering design should be accepted as furnishing a panacea for all the evils of quicksands and flood-waters combined, but, in preparing this paper on a phase of engineering which seemed to be enshrouded in obscurity, it was the writer's aim to present the elementary considerations which affect the design of all dams on alluvial foundations, without attempting to discuss those standard features of form, dimensions, stability against sliding or overturning, weep-holes or drainage facilities for hollow types of dams, and other matters which are fully treated in standard books.

Just as the simple formula, $v = \sqrt{2gh}$, may be modified by the introduction of variable coefficients (customarily designated as c and m) to adapt it to the calculation of the various conditions of flow of water throughout the entire range, from percolation through sand to flow in open channels, submerged orifices, spillways, sewers, pipes under pressure, etc., so may the elementary considerations, governing the design of dams upon alluvial foundations, be modified according to the special requirements of any set of local conditions.

Mr. Koenig.

Mr. Smith states that capillary attraction does not exist as a force in saturated soils, but, if capillarity be considered as a separate elemental force, the writer is of the opinion that it exerts a marked influence on the frictional resistance to flow through the interstices of alluviums.

If an impervious substance of irregular form, such as a large casting, be partly immersed in water or oil, there is a steady flow of a thin film of the liquid (of almost infinitesimal progress) upward, over, and around the object, until its entire surface is covered. In the case of water, evaporation into the atmosphere is so rapid that the film cannot extend very far above the surface, while, in the case of oil, which evaporates very slowly, the flow (or "creeping") of this film of liquid will continue over considerable heights and distances from the surface of the liquid.

For want of a better term, the force which attracts this flow, or "creeping," of a film of liquid, may be called molecular capillarity.

The effect of friction in retarding the velocity of flow of water, is generally considered as a negative, or obstructive, force which impedes progress, but, for some conditions of flow, as, for instance, pipes of very small diameter, it would seem logical to assume that frictional resistance to flow is largely a measure of this positive force of molecular capillarity, which causes a film of water to cling to the surface along the wet perimeter of the conduit.

If a new file, or a rough bar of iron which has not been insulated by a film of oil, be immersed in water and then suspended, it will be found that the film of water which clings to its surface is considerably thicker than that which clings to the surface of a bar of polished iron under similar conditions. The basis of contention, therefore, rests on the determination of whether this film of water which clings to such surfaces is sustained by frictional resistance, or by the force of capillarity which causes water to rise in a capillary tube to a higher level than that which surrounds it on the outside.

Capillary attraction, as a positive elemental force, is not confined to small circular tubes, but is just as marked in the case of two flat plates with a slight space between them and similarly immersed. Philosophers and scientists of various generations have propounded numerous hair-splitting refinements of reasoning on the phenomenon of capillarity, in their attempts to subdivide its effect into several elemental forces, such as capillary cohesion, molecular adhesion, and the surface tension of liquids.

The miniature tent-like mound of water which may be raised above the surface of a sheet of water, by the point of a Boyden hook-gauge, is attributed to surface tension. Similarly, if the sharp point of a pencil be brought into contact with the surface of a glass of water, and then raised, it will be noticed that a little mound of water is raised to appreciable height above the surface, before the water relinquishes its hold on the point of the pencil.

Mr. Koenig.

If the flat end of a glass rod, or an ordinary pencil, be carefully lowered vertically until it is in contact with the surface of a glass of water (Fig. 12), and is then slowly raised, the surface of the water will rise with it, in the form of a miniature mound. Then, as the rod, or pencil, is slowly raised higher, a column of water will rise with it, gradually reducing in diameter at the middle, like an hour-glass, until the height of the column becomes greater than the combined forces of capillary cohesion, molecular adhesion, and surface tension of the water can sustain; then it parts, leaving a spherical segment of water clinging to the end of the rod, or pencil.

The middle ordinate of this spherical segment of water is probably a measure of the distance from the surface at which the combined forces of capillary cohesion and molecular adhesion, between the particles of water and the sustaining surface, are effective to an extent which exceeds the force of gravity.

If the diameter of a capillary tube is so small that its hydraulic radius is less than the thickness of the film of water which is caused to cling to its surface (whether the force is designated by the name of friction or capillary attraction), the tube may be held in any position in free air, without causing appreciable movement or flow of the water through the tube.

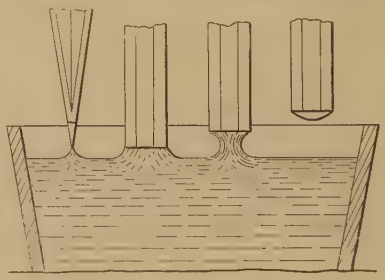


Fig. 12.

To cause a movement of the drop of water thus contained in such a small capillary tube, with appreciable velocity of flow, from one place to another in the tube, requires the application of positive force, thus indicating that, for this condition, the force of capillary attraction exceeds that of gravity.

With smooth glass capillary tubes of slightly larger diameters, the force required to produce a movement, or flow, of the drop of water is found to decrease inversely as the diameter of the tube, until the hydraulic radius becomes sufficiently large for the weight of the water to overcome the capillary resistance to flow. For diameters greater than this, one ceases to consider them as "capillary tubes" and begins to call them "pipes"; however, if we increased the diameters of capillary tubes and roughened their inner surfaces, the condition of effective capillary resistance to flow would continue for larger diameters than where smooth tubes are used.

If we must have "hair-splitting" refinements of terms to distinguish between the effects of friction and capillary attraction, as elements of the force which causes retardation of flow, then it will be necessary for some one to define them clearly, both in quality and magnitude, and to

indicate a line of demarcation which may show where one begins and the other ends. Mr. Koenig.

When water is poured on dry soil, it disappears more or less rapidly under the process designated by the term "absorption," and its downward progress through the interstices of the soil is generally attributed to the force of gravity alone. If we suspend a wick in such manner that its lower end is slightly immersed in water (or oil), the liquid which is "absorbed" by the wick flows upward until the wick is entirely saturated. This process, too, is ordinarily called "absorption," but it is absorption by capillarity, because absorption is a process and not a force.

If dry sand is slowly poured into one end of a trough which is partly filled with water, until the top of the sand is considerably above the surface of the water, it will be observed, after sufficient time has elapsed, that the line of saturation in the sand is somewhat higher than the level of the water in the trough, and that the entire mass of sand above the water surface has become moistened, except a thin layer on the surface from which the moisture is removed by evaporation as rapidly as it accumulates.

As capillarity is a positive force of sufficient magnitude in saturated soils to raise the level of saturation materially, it follows that it also exerts a corresponding influence in retarding the progress of an excess of water percolating through the soil by the force of gravity.

The statement of Mr. Price, that the construction of a dam which cannot be carried away by the force of flood-waters usually involves greater expense on sand foundation than where there is rock foundation, should be modified to the extent that it applies only to conditions of extreme variation in the ratio of the normal flow of the stream to the volume and depth of flood-waters. The broad, shallow rivers of the Western plains sometimes carry volumes of flood-waters many times greater than their minimum rates of discharge, with only from 5 to 10 ft. variation of depth.

The writer's observations have been principally confined to streams of this character, therefore he is pleased with this opportunity to express the hope that Mr. Price will amplify his discussion with specific analyses, accompanied by detailed sketches of his designs for the extreme conditions prevailing in the Atchafalaya River, at Simmesport, La. A dam which is founded on 80 ft. depth of fine sand, and successfully resists the destructive forces of flood volumes of 500 000 cu. ft. per sec., with fluctuations of 50 ft. in the depth of overflow on the crest, is an exceptional engineering achievement which is of vital importance to a general consideration of the subject in hand.

Mr. Price's statement of the requirements and special features of a dam on a sand foundation, cannot be questioned as effective means of securing permanence of construction under the extreme conditions

Mr. Koenig.

which he overcame so successfully. His method provides for conditions where it must be assumed to be impractical or impossible to check the undermining force of seepage flow by means which are described in the paper as effective for prairie streams, as such streams are subject to less violent flood conditions than those which obtain in the Atchafalaya River, and where, therefore, Mr. Price's method would seem to involve needless expense for periodical additional construction.

Mr. Potter calls attention to the remarkable development which has taken place in the Western States and the Canadian Northwest during the past decade, and the increasing municipal and commercial needs which make it necessary to harness the streams.

The average slope, or fall per mile, of these Western streams in alluvial soils, generally ranges from 5 to 7 ft. per mile, and the percentage of run-off which occurs after heavy rains over the porous watersheds results in flood discharges of much less volume than those which occur in sections of the country where steep, rocky slopes abound.

The progress of municipal and commercial growth in Western communities, is frequently marked by variable and uncertain "spurts," therefore it is sometimes deemed unwise to discount the future in proportioning engineering undertakings, by "guessing" at the probable trend of development and rate of growth, and attempting to make adequate provisions therefor. Questions of probable future enlargements, or the possible necessity for entire reconstruction, are frequently deferred for future solution, and it is demanded of the engineer that he restrict his designs to the scope and requirements of existing, known conditions, and that he provide for the future only to the extent of making low first cost the essential feature.

Pile-driving, in the sandy beds of Western streams, presents a variety of peculiar difficulties. Efforts to drive bearing-piles there, by the sheer force of long hammer-blows, will present as many peculiarities and uncertainties as may be experienced by the uninitiated while attempting to hasten the progress of a balky mule. Some piles may go down steadily, like those driven on land, while others will rebound at every blow, with such slight progress in permanent penetration, that the pile seems to be merely churning up and down like a ramrod.

In quaking sands, long hammer-blows seem to have little effect on the amount of penetration imparted to the pile, while considerable time and money may be wasted, because a large percentage of the piles will be split, broomed-up, or broken under such "punishment." A bearing-pile which has not reached the desired depth of penetration, yet which has apparently been driven "to refusal" under long hammer-blows of the specified intensity (as determined by the weight of the hammer and the length of the drop), may generally be "settled" slowly and steadily to the full depth of penetration desired, under the combined influence of a heavier hammer and cap and short jarring blows.

"Providence is always on the side with the heaviest artillery." Just so does successful pile-driving in quaking quicksand demand heavy equipment—and patience. It is necessary to weight the pile down with an extra heavy cast-iron cap, and then to attain the necessary intensity of blows by using a heavy hammer with short drops, permitting the hammer to rest on the pile for a few moments after each blow, while the reaction occurs in the quaking sand. Some deposits of fine sand, in the sand-bars of these streams, seem to have a rubber-like consistency, so that when the combined weight and short jarring blow of the heavy hammer cause a pile to sink a little, the sand surrounding the pile is depressed with it, and then slowly adjusts itself as if creeping upward around the pile and swallowing it. Mr. Koenig.

The difficulties which attend the driving of wooden sheet-piling, under such conditions, are too obvious to require description.

Deposits of large boulders are rare in the sand-beds of Western streams, except where extensive deposits of glacial drift occur, therefore there are few difficulties to contend with in driving piles (either bearing- or sheet-piles) by the jetting process. When a pump of sufficient capacity is provided, the jetting process of pile-driving in sand is so simple and effective that one is moved to wonder why it is so seldom used. Instances where it has been tried and condemned as unsatisfactory have generally been due to the use of small pumps of insufficient capacity, and too much reduction, or "choke," at the nozzle end of the pipe. A slight reduction of area at the end of the pipe will be sufficient for the purpose of a nozzle, because the volume of water (within reasonable limits) is more essential than the velocity of flow of the jet issuing from the nozzle, in the attainment of both speed and accuracy of alignment, in sinking the piles.

In Western alluvial formations, which do not contain large boulders, steel sheet-piling may be successfully and satisfactorily driven to almost any desired depths which are likely to occur within the range of practical requirements, but, from observations on submerged steel structures, many engineers are of the opinion that steel sheet-piling is not a desirable material for use as a permanent curtain-wall under a dam.

There have been numerous examples of the perfect alignment which may be attained with steel sheet-piling in the construction of deep coffer-dams; instances where the alignment was so true that the last, or closing pile, which interlocked with an adjacent pile on each side, was driven down without apparent distortion, or undue strain. At St. Louis (it is claimed), such sheet-piles cut through some old willow mattress-work, severing the large steel cables (old street-railway cables) with which the various sections were bound together, and cutting, crushing, or displacing the large pieces of rock and other material with which the mattresses were weighted down. At Omaha,

Mr. Koenig.

the steel sheet-piling of a coffer-dam in the Missouri River was driven to a penetration of several inches into the bed-rock, without apparent injury, for the piles (Freystedt pattern), upon being withdrawn, showed no indications of distortion or battering at either end.

In view of the examples of perfect alignment attained with the use of steel sheet-piling, and the recognized degree of uncertainty which attends the attainment of perfect alignment in the construction of a curtain-wall of the more desirable wooden sheet-piling, at considerable depths, it would seem to be both practical and economical, for any important structure where a permanent curtain-wall of submerged sheet-piling is required, to utilize a section of temporary curtain-wall of steel sheet-piling, driven to the desired depth of penetration, as a guide, in jetting-down the permanent wooden sheet-piling, thus ensuring both perfect alignment and water-tightness.

Steel sheet-piling may apparently be driven, pulled, and re-driven, in sand, many times without injury, therefore a long curtain-wall of permanent wooden sheet-piling might be accurately placed by using 40 or 50 lin. ft. of steel curtain-wall as a guide, and pulling and re-driving the steel piling a little in advance of the construction (jetting-down) of the wooden sheet-piling.

The depth to which it may be found practical to jet down dove-tailed or interlocking wooden sheet-piles in sand, along the face of a guide-wall of steel sheet-piles, with immunity from injury, and with precision of alignment, is apparently limited only by the depths which it is practical to attain by fastening "follow-up" lengths of steel sheet-piling together in such manner that they may be driven, pulled, and re-driven innumerable times without injury.

Mr. Thomson's reference to the erratic performances of certain steel sheet-piling driven at the Brooklyn Navy Yard, and other instances that have come under his observation, would, if amplified by further definite information, undoubtedly prove interesting to many; especially if the bending, twisting, and coiling of the bottoms of those piles were not directly attributable to large granite boulders, or other unusual obstructions.

The success which has attended the use of steel sheet-piling in the alluvial soils of the West, is such that the average Western engineer would contemplate the use of lengths (jointed) considerably in excess of 50 ft., without the slightest hesitation, unless such piling were to serve merely for temporary structures, and economy made it necessary to consider the question of salvage.

Mr. Potter suggests that unequal settlement is another cause of frequent failure in dams on alluvial foundations, and, that in structures of considerable magnitude, it is impossible to prevent unequal settlement, because of the non-homogeneity of the compressible foundation. The writer, however, is pleased to take issue with him on these

points, as regards structures of average magnitude, with the hope of provoking discussion from other members whose personal experience and observations may enable them to contribute information on the subject. Mr. Koenig.

Few materials in their natural formations (from ordinary loam to shales and rocks) possess the degree of homogeneity (with reference to incompressibility) which is characteristic of the deep deposits of sand which form the beds of many Western streams. A body of rigidly confined sand, which has been agitated and then settled by saturation with water, possesses a degree of uniformity and incompressibility which makes it an almost ideal material for foundation purposes, as long as it is thus properly confined; but, on the other hand, no other material is more treacherous than sand which is not properly protected against lateral movement, or the disintegrating effect of lateral percolation.

The writer is of the opinion that, for dam or diversion weirs of average magnitude, on sand foundations, under the conditions which prevail on average Western streams, no settlement-joints, or other provisions for flexibility, are required, or even desirable, if the sand foundation under the structure is adequately protected against disintegration from the effects of lateral seepage flow, by using a tight curtain-wall of sufficient depth of penetration, so that the velocity of the seepage flow which finds its way under such curtain-wall is retarded to less than that which will cause the movement, and subsequent washing-out, of even the finest particles of the alluvium.

With this accomplished, the next step is to provide adequate means to prevent the erosion of the surface of the river-bed, for a reasonable distance below the dam, by the swirl of the water discharged over the spillway.

Frizell (in "Water-Power") states that the tremendous energy acquired by water falling over the crest of a dam is broken by eddies and counter-currents near the toe of the structure, and that the violence with which the water is agitated gradually diminishes, until at a down-stream distance of about four or five times the height of over-fall, its force seems to have been entirely dissipated, and the stream presents its normal conditions of flow.

The writer's design provides a water-cushion at the toe of the dam, entirely submerged below the bed of the stream, to absorb the shocks of the water, ice, and débris falling over the crest of the spillway. The outer edge, or lip, of the trough forming this water-cushion is shown to be finished off a little below the normal level of the bed of the river, and the down-stream width of the trough is sufficient to include the swirling counter-current, or vortex, resulting from the abrupt change in the direction of flow. The undissipated energy remaining in the water after it has passed beyond the lip of the

Mr. Koenig.

water-cushion is prevented from eroding the bed of the river (within this "critical" distance equal to about four or five times the height of over-fall) by a substantial concrete curb, placed at the level of the normal bed of the stream and supported by a line of tight sheet-piling, at about that distance below the dam.

Ordinarily, the erosive force of the water flowing over that confined section of the natural river-bed will not exceed that which would be required to lift the grains of sand vertically even a fraction of an inch, but, assuming that during periods of flood, some erosion may occur in this section, it will merely result in scouring out a few inches depth of sand, between the edge of the trough and the concrete curb, thus providing a shallow water-cushion over the surface of the sand, within which the agitation of the water flowing from the trough, and the upward flow of the seepage-water passing under the main curtain-wall, may cause particles of the sand to "dance" around, without, however, exerting sufficient force to lift them through the vertical distance necessary to carry them over the curb, while even this slight effect may be entirely eliminated by spreading over this section coarse gravel or broken stone, where such are available.

Thus, for the conditions prevailing in average Western streams, adequate provision is made for all the forces which are likely to cause the destruction of a dam on a sand foundation.

In rivers having slopes exceeding 5 or 6 ft. per mile, the erosive force of the high normal velocity of flow may necessitate special provisions to ensure the permanence of the concrete curb (the secondary dam) with its supporting line of sheet-piling, because a gradual scouring-out of the bed of the river below this "sub-dam," without the replacement with sand which formerly washed down stream from points above, might soon convert the curb into a low weir, with an appreciable height of over-fall, which would then gradually increase until the "sub-dam" and its supporting line of sheet-piling were eventually undermined and destroyed.

The most effective means of preventing this, would probably be attained by making the sub-dam of heavier cross-section and extending it down to a depth of about 6 or 8 ft. below the normal bed of the stream; then, on its down-stream side, there should be provided a series of concrete slabs, constructed with flexible connections and anchorage (as suggested by Mr. Price and others), which will permit the apron to settle gradually as the scouring out of the sand proceeds, without endangering the concrete curb and its line of sheet-piling, which are such vital elements in the safety of the main structure.

With careful workmanship and inspection, it is entirely feasible to construct a tight cut-off wall of wooden sheet-piling, to the required depth of penetration in sand, under the heel of a dam; and its efficiency in retarding the velocity of flow of the seepage-water at the

toe of the dam is beyond question. It is equally true, however, that a line of tight sheet-piling across a stream which traverses a broad valley underlaid with a deep stratum of sand, has no appreciable effect on the general movement of the underground waters in the valley. Mr. Koenig.

This was amply illustrated, during the early Nineties, when a water-power development from the Platte River, with about 20 miles of canal, was constructed at Kearney, Nebr. A sheet-pile dam, or diversion weir, was placed across the entire channel of the river, at an up-stream angle from the head-gates, for the purpose of obstructing (or intercepting) the under-flow sufficiently to raise the level of water to a height of about 6 ft. above the bed. The upper reaches of the Platte River, in the extreme western portion of Nebraska, flow perennially, but, a few miles farther east, the surface flow, during the dry summer months, entirely disappears in the sand and joins the great body of ground-water which underlies almost the entire State of Nebraska, just a few feet below the surface. The construction of a line of tight sheet-piling ½ mile or more in length, and with a depth of about 16 ft. below the bed of the river, as carried out at Kearney, apparently had no effect whatever on the progress of the under-flow, therefore it was found necessary to abandon the dam and to continue the canal farther up stream, where deep seepage basins were constructed for the accumulation of the ground-waters to supply the canal.

Mr. Smith suggests that the sheet-piling should not be considered as an independent feature of the design, and that the same result can be attained by a long base with even no piling at all, or by a short base with piling.

The paper emphasizes the fact that the base width, or the width of the apron below the dam, is an important element in fixing the practical factor of safety of a dam on a sand foundation; while, the desirability of a liberal application of coarse broken stone, spread in the order of inverted Telford, over the surface of the river-bed for some distance below the dam, is so obvious that it seemed unnecessary to mention it for localities where cheap stone is available.

The rigid economy in the use of broken stone, which was observed in the design illustrated in Figs. 1 and 2, in which not even the short space between the edge of the trough forming the water-cushion, and the concrete curb, or sub-dam, is thus protected, is based on the fact that along many of the important streams in the Western States, and in the Canadian Northwest, rock is so scarce that crushed stone costs from $3 to $5 per ton. The mercenary spirit of modern commercialism is very much opposed to a promiscuous scattering of such "nuggets," and the territory which was thus overlooked by the Creator, in the distribution of this desirable material, comprises a very large proportion of the Western States and Canada, in which a great many diversion

Mr. Koenig.

weirs and dams will be constructed for purposes of irrigation works and hydro-electric power developments.

The writer is of the opinion that two lines of water-tight sheet-piling, with proper depths of penetration and relative positions, substantially as illustrated in Figs. 1 and 2, for the purpose of regulating and controlling the underflow, are the essential considerations to be observed in designing dams on alluvial foundations, to ensure permanence of construction, and at the same time to meet the demands for economy in the first cost of such structures. Rip-rapping, and the lavish use of coarse broken stone, are useful, desirable, and even indispensable, under certain conditions, but they are of less vital importance to the permanence of such structures.

In constructing a conduit, or pipe-line, through an earth dam or reservoir embankment, it is necessary to provide flanges, or offset projections, at intervals, for the purpose of breaking joints with the earthwork, otherwise water will creep along the straight sides of the masonry or pipes, and gradually undermine the embankment. Similarly, if an alluvial embankment is to be constructed on a rock surface, it is necessary to provide a masonry core-wall to prevent the water from creeping along the surface of the rock, and cutting a channel which will eventually destroy the embankment.

If, then, this order is reversed, and a concrete dam is placed on the surface of an alluvial deposit, it is just as essential that a curtain-wall be provided for the same purpose. The writer's opinion of the relative importance of the base width and the depth of penetration of sheet-piling, or other form of curtain-wall, in retarding the destructive force of underflow, is, that if the value of the base width of the dam for this purpose be assumed to be in direct proportion to such width, within reasonable limits, then the value of the curtain-wall would be about as the square of the depth of penetration, within the reasonable limits of practical requirements.

That an adequate knowledge of the laws of the flow of water through sands and gravels is of fundamental importance, in investigating the subject of stability of dams built on such foundations, as Mr. Hazen states, was recognized by the writer at the beginning of his analysis of the subject, but, in endeavoring to harmonize existing data and formulas for the determination of the velocity of flow through sands and gravels of various degrees of porosity, with a view of establishing a practical, safe, and convenient rule for the determination of the probable destructive force of seepage flow, the uncertainties which would affect the accuracy of such calculations became so apparent that the writer was moved to reverse his original policy and devote a large part of the paper to the discussion of the impracticability of endeavoring to apply, to the natural sand-beds of rivers, those reasonably exact formulas devised by Mr. Hazen for the determination of the

velocities of flow through sands and gravels of known degrees of porosity. Mr. Koenig.

Below the surface stratum of sand, which is intermittently shifted from place to place by variations of the velocity of the current, the natural deposits of sand and gravel forming the beds of many Western streams are composed of strata of sand and gravel which form a homogeneous mass, as regards its specific gravity, its incompressibility when confined, and other properties which would affect its suitability and bearing value for foundation purposes; but, strata of various degrees of porosity alternate with such irregularity, that a wide margin of safety must be provided for, in making calculations of the probable maximum, minimum, or average velocity of seepage flow.

It is manifestly impractical to attempt to make sufficient borings or soundings for an exact determination of the depths and widths of the various strata of sand and gravel, of various degrees of porosity, across the entire width of a stream at the site of a proposed dam, and the writer's analysis indicated that such information would have little real value, even if it were practical to obtain it.

In determining the theoretic minimum depth of penetration required for sheet-piling, in terms of the head, h, the writer's formula includes both the porosity and specific gravity of the material to be penetrated, as factors; therefore it may be applied to any conditions or class of soil for which the porosity and specific gravity are known. The variation of the results obtained under various assumed conditions, however, indicates that no great degree of refinement is necessary for purposes of determining the depth of penetration required for sheet-piling, which is the primary purpose for which the formula was devised.

It was shown that for sand having a porosity of 40% and a specific gravity of 2.65,

$$d = \frac{h}{2.65 \times 0.6} = \frac{h}{1.59} = 0.629\ h.$$

Assuming for clay a porosity of 20%, and a specific gravity of 1.90,

$$\text{then } d = \frac{h}{1.9 \times 0.8} = \frac{h}{1.52} = 0.657\ h.$$

Assuming a clay having a porosity of 40%, and a specific gravity of 1.90,

$$\text{then } d = \frac{h}{1.9 \times 0.60} = \frac{h}{1.14} = 0.877\ h.$$

Assuming for loam a porosity of 30%, and a specific gravity of 1.90,

$$\text{then } d = \frac{h}{1.9 \times 0.7} = \frac{h}{1.33} = 0.75\ h.$$

The specific purpose of this formula is not that of determining the volume of seepage flow which will find its way under the curtain-

Mr. Koenig.

wall, but to determine a depth of penetration which will ensure a reduction of its velocity of flow along the base of the dam, below that which will cause a movement or washing out of the finest particles of the material. For this purpose, the formula is rational, because for light, fine-grained soils having a low specific gravity, it indicates greater depths of penetration than for heavy materials. For materials of equal specific gravity, from sand to coarse gravel or broken stone, it indicates the same depth of penetration, but, the frictional resistance to flow through the interstices of the material is inversely proportional to the effective size of the interstices, therefore a very low velocity would result through fine sand, while the relatively higher velocity which would result through the interstices of the same depth of coarse gravel or broken stone, is automatically compensated for by the fact that the pieces of gravel or stone will stand a much higher velocity without movement or displacement.

Mr. Hazen's account of the successful application of his formula for the velocity of seepage flow, in determining the size of the canal and the pumping equipment required to drain the site of the filter plant at Toronto, Ont., prior to construction, is both interesting and instructive. In view of his extensive experience and successful achievements in the designing and construction of filter plants and other works involving the principles of seepage and percolation, it is to be hoped that he will contribute further practical information on the subject, not only with a view of analyzing the laws of seepage flow under curtain-walls of various depths, but also a variety of practical experiences with various natural conditions such as those at Toronto.

Some 15 or 20 years ago, the seeming impossibility of constructing permanent dams on the sand foundations of Western prairie streams gave rise to a number of irrigation projects, for which the supply of water was to be secured from seepage basins, excavated along the banks of streams to considerable depth below the level of saturation in the soil, from which it was to be conducted away in canals for irrigation purposes.

There seems to be no published information or data on the practical results achieved by these undertakings, and, while they may not have been entirely satisfactory, or financially successful, in many cases, definite data on the subject are of increasing importance, because unirrigated lands still exist in territories where, not only the normal flow of the streams has been appropriated, but also considerable proportions of the storm flood-waters are conserved in storage reservoirs.

If the question of recovering return seepage waters in the lower benches, or along the banks of a stream, below large irrigation works, can be determined with reasonable accuracy, then perhaps some of the lower laterals of an extensive canal system might be supplied, or have their flow augmented, from such sources, so that a larger area of

territory may be served by the quantity of water which enters the main canal at the head-gates. Mr. Koenig.

The increased cost involved by such a system would preclude its voluntary adoption for private undertakings, but, water being a precious commodity in some sections of this country, economic considerations, at some future time, may make such measures necessary, if it can be shown that it is feasible to utilize the sub-drainage from one series of irrigation benches to supply water to lower benches, beyond the territory which can be served by direct application of water from the upper system of canals and laterals.

The writer can claim no extensive experience in the actual construction of dams. He has had occasion to design several which, for one reason or another (generally lack of funds), were never built, and on several occasions has had to refuse his professional services in cases where the restrictions were such that he preferred not to be a party to the attainment of results which would prove unsatisfactory to himself; therefore, he has no failures to his discredit.

The design illustrated in Fig. 1 is intended for the 600-ft. spillway section of a dam in the Loup River, in Nebraska, for a hydro-electric development having a continuous capacity of 30 000 electrical horsepower. The remainder of the dam will consist of 5 700 lin. ft. of earth (sand) embankment, connecting with one end of the spillway section, thence diagonally down stream along an island to a short concrete dam, 100 ft. long, which provides a spillway for the discharge of ice and débris, submerged sand-gates, or scouring sluices, and at right angles to this short dam will be the head-gates for the admission of 3 200 cu. ft. per sec. to the canal.

A history of the dams which have been constructed, or attempted, in Nebraska streams, would be principally a chronicle of failures, because very few of the many attempts which have been made have resulted in success. In the Elkhorn River at Stanton, in the Cedar River at Erickson, and at other places, three or four attempts were made in succession, at the same sites, but, in each case the dam failed within a few days, weeks, or months after its completion.

Each instance of failure in engineering works, exerts a deterrent influence on future undertakings of a similar character, therefore the writer offered this paper in a frank spirit of endeavor to stimulate discussion of the subject, and the desire to collect, in the *Transactions* of this Society, a variety of information, opinions, experiences, and successful designs, from engineers in various parts of the country, which would enable engineers in the future to attain a greater measure of success in their efforts to subdue the treacherous dragon, "Quicksand."

Frizell (in "Water-Power") devotes a chapter of about twenty pages to descriptions of notable failures of high dams which have occurred from the beginning of the last century down to those which

Mr. Koenig.

have horrified the present generation. This chapter alone is valuable to every engineer, and, one cannot read it without regretting that Mr. Frizell did not extend it to several times as many pages, by including descriptions and analyses of failures of lesser magnitude and less destructive consequences.

The booklet to which Mr. Beardsley refers,* in which the general principles of dam construction were completely elucidated, while the "gravity" dam, as patented by E. R. Beardsley in 1894, is described as the only panacea against all the ills which cause the destruction of other types, is certainly a unique creation. In that booklet, the causes which brought about the failure of the Austin Dam (at Austin, Tex., in April, 1900) are explained in a positive, off-hand, manner which puts to shame the efforts of Frizell and other eminent engineers to analyze them. In the same manner Mr. Beardsley now expresses his off-hand opinion that the design illustrated in Fig. 1, would be sure to fail. He states that two dams, built like that shown, failed, therefore, either he has missed the essential features of the paper entirely, or it is to be assumed that he anticipated the writer's ideas.

In "Hydro-Electric Plants," he devotes considerable space to illustrations and descriptions of the "Beardsley mat" and the "Beardsley gravity dam" for alluvial foundations, without citing specific instances of successful applications of these "patented" features. Several attempts to construct Beardsley dams in Nebraska and Iowa streams were lamentable failures, and the writer knows of none which was constructed on sand foundation which has resulted in success; therefore he considers that portion of Mr. Beardsley's book on "Hydro-Electric Plants" positively dangerous in the hands of inexperienced young engineers, who might become impressed, and attempt to design important structures in accordance with the principles and special features recommended.

Darcy's law of the movement of ground-waters was determined from a series of experiments with columns of soil of various depths, or lengths, under various pressure heads. He found that the velocity of flow varied directly as the pressure head, and inversely as the depth, or length, of the soil column.

His conclusions are expressed in the following formula:

$$v = c\frac{h}{l} \qquad \text{(Darcy)}$$

In which v = Velocity of moving ground-water;
h = Head, or difference of pressure at the two ends of the soil column;
l = Length, or depth, of the soil column;
c = A constant, depending on the character and porosity of the soil.

* "The Gravity Dam," by E. R. Beardsley.

Mr. Koenig.

Hazen's exhaustive experiments substantiated Darcy's law, but they also resulted in a formula which permits greater refinement of calculation, through the introduction of factors representing the "effective size" of the soil grains and the temperature of the water. He found that the velocity of flow increased with higher temperatures, and also that it increased about as the square of the effective size of the soil grains.

His complete formula therefor is as follows:

$$v = c\, d^2\, \frac{h}{l}\, (0.70 + 0.03\, t) \ldots\ldots\ldots\ldots \text{(Hazen)}$$

In which v = Velocity of flow, in meters daily;
c = A constant (approximately 1 000);
d = Effective size of soil grains, as determined by Hazen's standard method of gauging;
l = Length of soil column, or thickness of sand-bed;
t = Temperature of water, centigrade scale.

Slichter* devised a formula, for determining the flow of water through sand, which takes account of the friction between the particles of water, known as the coefficient of viscosity, and is defined as the amount of force necessary to maintain unit difference in velocity between two layers of water at unit distance apart.

His complete formula is:

$$q = 0.2012\, \frac{p\, d^2\, s}{\mu\, h\, K} \text{ cu. ft. per minute} \ldots\ldots\ldots \text{(Slichter)}$$

In which q = Cubic feet of water transmitted through the column of sand per minute;
p = Difference in pressure at ends of column, or head under which the flow takes place, measured in feet of water;
d = Mean diameter of soil grains in millimeters, or the "effective size";
s = Area of cross-section of sand column, in square feet;
μ = Coefficient of viscosity of water, which varies with the temperature of the water;
h = Height of soil column, in feet;
K = A constant depending on the porosity of the sand.

According to the author's analysis, the theoretic minimum depth of penetration required for sheet-piling, as a curtain-wall under a dam on sand foundation is:

$$d = 0.629\, h \ldots\ldots\ldots \text{(Theoretic minimum)}$$

In which d = The depth of penetration below the bed of the stream; and
h = The effective head of water impounded, or the difference in level between the water above and below the dam.

* "The Flow of Underground Waters." U. S. Geol. Surv., Water Supply Paper No. 67.

Mr. Koenig.

Let us take, for example, a low dam which impounds an effective head of 10 ft. ($h = 10$), and assume the extreme conditions (which are not likely to occur in Nature) that the bed of the stream is composed of coarse sand and gravel of an effective size of 1.0 mm., and that the temperature of the water is about 25° cent. (or 77° Fahr.). Then the velocity of seepage flow under the bottom of the curtain-wall of sheet-piling of the theoretic depth of penetration ($d = 0.629\ h$) or 6.29 ft., according to Hazen's formula (assuming free escape at that point), will be:

$$v = 1\,000 \times 1.0 \times \frac{10}{6.29}\,(0.70 + 0.75)$$

$$v = 2\,204 \text{ meters} = 7\,227 \text{ ft. per day,}$$

$$\text{or } v = 0.0836 \text{ ft. per sec.,}$$

or, approximately, only one-third of the velocity which, according to DuBuat's experiments, is required to move fine particles of potter's clay along the smooth bottom of a wooden trough.

The effective length of the column of sand which resists the flow of water under the curtain-wall from the up-stream side, along its path of flow to the toe of the dam, in Fig. 1, is approximately 2.5 times the depth of penetration of the sheet-piling, therefore the velocity of flow of the seepage water issuing under the toe of the dam will be reduced to:

$$v = 0.035 \text{ ft. per sec.,}$$

according to Hazen's formula, thus indicating that the writer's formula provides an ample margin of safety against the movement of even the finest particles of silt, or clay, by the force of seepage flow.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1197

STREET PAVING CROWNS, WASHINGTON, D. C.

By T. J. Powell, Assoc. M. Am. Soc. C. E.

With Discussion by E. A. Steece, M. Am. Soc. C. E.

The writer has read with much interest the paper by P. E. Green, Assoc. M. Am. Soc. C. E., entitled "A Review of Chicago Paving Practice,"* and also other recent papers† on this subject, and submits the following:

There seems to be a great difference of opinion among engineers on this subject, and, as Mr. Zahniser says, a collection of these formulas may be the beginning of a possible "Rational Formula."

The following method of treating crowns has been used by the Engineer Department of the District of Columbia since 1894, but has only recently been formulated.

This formula was suggested and deduced by Mr. Joseph W. Dare, Assistant Engineer of the District of Columbia, and takes into account the width of the roadway and the longitudinal grade of the street. It is applicable for all widths of roadway up to and including 50 ft., after which it is necessary to treat the section as a special one. The formula is:

$$C = \frac{W\ (100 - 4\ P)}{6\ 300 + 50\ P^2}$$

* *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 1.

† In *Engineering News*, by Mr. George C. Warren, entitled "Paving Practice with Respect to Crowns of Roadway Pavements and Concrete Foundations," December 2d, 1909; by Mr. G. B. Zahniser, entitled "Suggestions for a Rational Formula for Street Pavement Crowns," May 5th, 1910, and by James N. Hazlehurst, M. Am. Soc. C. E., entitled "Formulas for Determining Street Crowns when the Two Curbs are at Different Elevations," June 30th, 1910.

In which C = the crown, in inches,

P = the longitudinal grade, expressed as a percentage,

W = the width of the roadway, in inches.

When the curbs are level, the crown is distributed as shown by the following formula:

$$\frac{8\,C}{0.3\,R} = d$$

$$a \text{ or } b + \frac{C}{18} = \text{the elevation at } A \text{ or } D;$$

$$a \text{ or } b + \frac{C}{12} = \text{the elevation at } B;$$

d = the transverse grade, expressed as a percentage;

a or b = the elevation at the gutters, expressed in feet and hundredths.

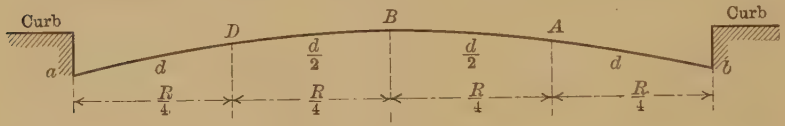


Fig. 1.

As will be noticed in this formula, the percentage of grade from gutter to quarter and from quarter to crown in no case exceeds 4% and 2%, respectively, and the diagonal rate from quarter to the curb, along the hypotenuse of an isosceles triangle, the legs of which are equal to the distance from the quarter to the curb, will not materially exceed the longitudinal rate, thus keeping a team on the same rate of grade, whether they are going straight up hill or taking a diagonal course as is their desire if left to themselves.

This, as far as the writer knows, is the only formula in which this holds good.

Another point, which has been taken into consideration and is shown in the following formulas, is the location of the crown when there is a difference in the elevations of the curbs.

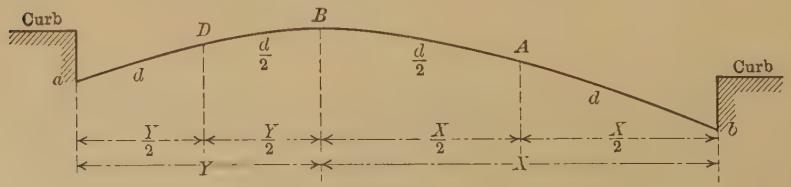


Fig. 2.

$$\frac{a-b}{1\frac{1}{2}\,d}+\frac{R}{2}=X$$

$$R-X=Y$$

$b+\frac{X}{2}\,d=$ the elevation at A;

$b+\frac{3}{4}\frac{X}{}\,d=$ the elevation at B;

$a+\frac{3}{4}\frac{Y}{}\,d=$ the elevation at B;

$a+\frac{Y}{2}\,d=$ the elevation at D;

X = the long side of the crown;
Y = the short side of the crown;
R = the width of the roadway, in feet and hundredths;
d = the transverse grade, expressed as a percentage.

This formula puts the crown and quarter points in such a position that the transverse grades from gutter to quarter and from quarter to crown will be the same as if the curbs were level, for the same longitudinal grade.

It is the practice in Washington, D. C., to put vitrified block gutters on streets on which the longitudinal grade is 1.5% or less. This does not change the formula, as 0.1 ft. is added to the curb side of the gutters for the rise of the same. The crown is then worked, using the distance between gutters as the width of roadway.

These formulas refer particularly to streets paved with sheet-asphalt, but for asphalt block, granite block, or other pavements having a more or less rough surface, and therefore giving a more secure foothold, it can be used equally well and give as good results by the addition of 1 in. to the amount of crown given by the above, This gives rates a little steeper from gutter to quarter and from quarter to crown, which is necessary, as these materials when paved on a gravel or sand base, as is usually the case, have a tendency to settle until all joints are entirely closed and water does not get between them.

These crowns give transverse grades sufficient to carry all water to the gutters as rapidly as necessary, besides reducing to a minimum the number of accidents caused by horses falling; and it produces a section which is pleasing to the eye.

DISCUSSION

Mr. Steece.

E. A. Steece, M. Am. Soc. C. E. (by letter).—In the consideration of formulas for street paving crowns, as suggested by Mr. Powell, it is the writer's opinion that a complicated formula is of doubtful utility. The crown is made, primarily, for water parting, and, except for this, the advantage of the inclination formed at intersections, sharp curves, and possibly esthetic considerations, the roadway would have a plane surface transversely.

There have been, and, in all probability there will be, many suggestions for so-called rational formulas—differing somewhat from uniform ratios—for determining the crown. It appears that the first impulse of many highway engineers is to derive a formula to cover this vexed question; it also appears to be simple, at first glance, to write a good one, but the further it is developed the more unwieldy it becomes, until, finally, something almost as complicated as Kutter's hydraulic formula (if rational) is produced. Evidently, any formula which will not serve beyond the limits of common practice is not rational.

The formula attributed to Mr. Dare:

$$C = \frac{W\ (100 - 4\ P)}{6\,300 + 50\ P^2}$$

is a modification of that of the late A. Rosewater, M. Am. Soc. C. E.:

$$C = \frac{W\ (100 - 4\ P)}{5\,000}$$

or *vice versa,* in which

C = the crown, in inches,
W = the width of the roadway, in inches, and
P = the percentage of grade.

Both these formulas produce lessening crowns on increasing grades.

The writer, in a limited practice, has observed that for impervious surfaces, such as asphalt, brick, etc., a crown of from 1 to 1.25% of the distance between curbs, for any grade, will come about as near to meeting the requirements of drainage and security to travel as any. For semi-pervious surfaces, which necessarily have an element of roughness, the crown should be made from 1.5 to 2.25% of the width of the roadway.

Such rules may be considered as too indefinite, but the engineer should exercise his judgment, as the conditions may warrant, for in this, as in every other branch of engineering, judgment is the prime necessity of good design, and cannot always be reduced to formula.

Mr. Steece.

The writer is not inclined to view with much favor the plan of shifting the crown from the center of the roadway on streets having one curb lower than the other, as this not only extends the transverse slope on the lower side—augmenting an existing evil without accomplishing anything, except, in some instances, preventing the surface water from flowing across the roadway—but the unsymmetrical effect is unsightly.

The contention that the crown should be lessened on the steeper grades seems to be open to argument, also, notwithstanding the fact that eminent municipal engineers have decided that this should be done. To lessen the crown, in this instance, is to reduce the transverse slope to a minimum. Admitting that this feature is most jeopardizing to travel, it must be admitted, also, that it has its greatest effect where the speed of travel is greatest, which is most apt to be on the minimum grade. In cases where the crown is greatly reduced on steep grades, the gutters become too shallow to prevent storm-water from intersecting streets from flooding out to the crown and scattering débris over the roadway. While this is partly a storm-water and street-cleaning problem, these factors should receive their share of consideration.

If all elements are to be considered in determining street crowns, a degree of complication is produced which will hardly be susceptible of formulation. Withal, it is one of those subjects which are not small enough to leave nor large enough to bother with, but which recur just often enough to tantalize the engineering mind.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1198

BOND-FRICTION-RESISTANCE IN REINFORCED CONCRETE.*

By William Fry Scott, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Edward Godfrey, Joseph T. Maguire, A. N. Talbot, Hector Robins Burroughs, Albert I. Frye, F. E. Turneaure, and William Fry Scott.

Early in 1907 the writer began a systematic study of bond in reinforced concrete in flexure. This study was particularly directed to the published reports of experiments by eminent American investigators on reinforced concrete. The reports on hand at the time the study began were by Professors Hatt, Lanza, Talbot, Howe, Turneaure, and Marburg, and Messrs. Humphrey, Harding, and others. Later, further reports were obtained from the Universities of Illinois and Wisconsin. The files of *Engineering News, Engineering Record,* and *Concrete Engineering* were carefully scrutinized, and other publications were examined for data on this subject.

As a result of this investigation the writer has proven that the usual assumption that reinforced concrete is "practically a homogeneous material" is not true, and it follows from this that sectional planes before bending do not remain plane surfaces after bending.

The method of analysis pursued by the writer, though not along strictly conventional lines, as laid down in the current literature on the mechanics of a reinforced concrete beam, was nevertheless mathematically correct. A reinforced concrete beam is not like a timber girder or a steel beam. It is a really wonderful combination of two unlike materials, each having properties when alone that are not at

* Presented at the meeting of December 7th, 1910.

all in evidence when the two are combined. Further, there are no precedents in the history of the mechanics of materials for such a combination.

The following is a brief synopsis of the mechanics of the beam as adapted to this study:

The bending moment produced on a beam by external forces is resisted by an equal and opposing moment which is developed in the beam—this is the section moment. If a transverse section is made through a beam, the action of the portion to the right on the portion to the left may be reduced to two equal forces, one positive and the other negative, which two form a couple the arm of which is the distance between their respective points of application. The point of application for the former will be the centroid of the compressive stresses; and of the latter, the center of the steel. Manifestly, the force acting on the steel in one direction, say, to the right, is equal to the friction force operating to the left between the steel and the surrounding concrete. This latter force is usually called "bond," but, as the real bond is broken at a very low stress compared with the final resistance at failure, and, as a kind of friction-resistance ensues after the bond is broken, it has been considered better to call it bond-friction-resistance. This bond-friction-resistance, therefore, may be defined as a friction force which operates after incipient movement has taken place. The intensity of the tensile stress that can be developed in the steel at a cross-section is mainly dependent on this bond-friction-resistance.

The following formulas have been used:

$$M_b = M_s = f_s A_s d' = f_b \Sigma O l' d',$$

in which M_b is the bending moment,

M_s is the section moment,

f_s is the unit stress in the steel,

A_s is the total area of the cross-section of the steel,

d' is the distance from the center of the reinforcement to the centroid of compressive stresses,

f_b is the average value for bond-friction-resistance per unit area of surface of rods to, say, the left of the cross-section;

ΣO is the sum of the perimeters of either round or square rods, and, l' is the entire length of the encased portion of the rod to the left of the cross-section.

The following is a short history of the various steps leading to the final results that have been developed from this investigation:

The first phenomena noted were: that perfect bond between the concrete and the surface of the steel did not seem to be very lasting; that, after the real bond was broken, there was a creeping of the rods accompanied by a frictional resistance between the particles adhering

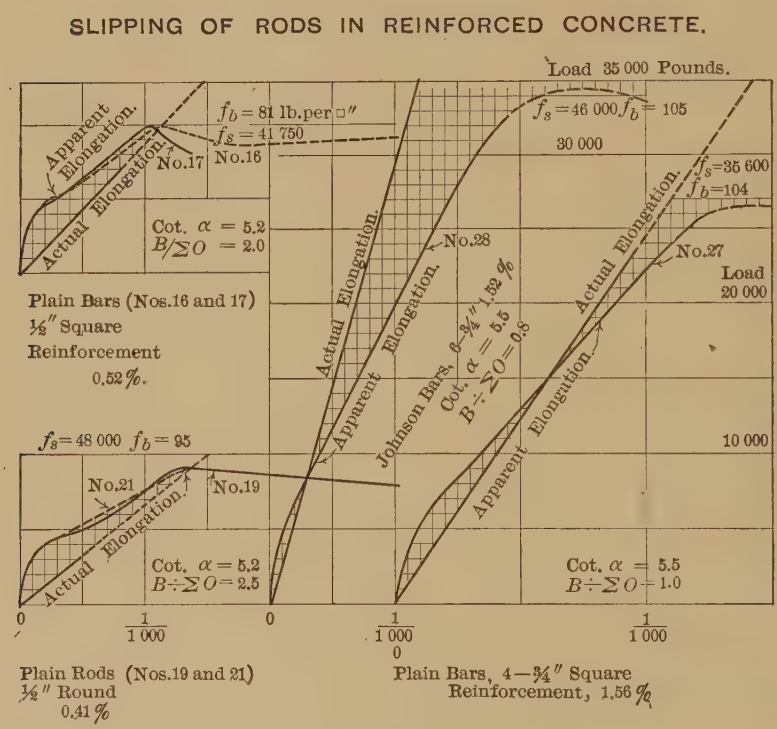


FIG. 1.

to them and the rough surface of the concrete encasing them; and, that, in cases of repeated loading, after the load had been taken off, the rods would not always return to their original position—evidently having been stopped in their return because loosened particles of the concrete turned sufficiently to wedge in and prevent it.

The slipping of rods is shown on Figs. 1 and 2. The group of beams in Fig. 1 is from Talbot's series of 1904. The group in Fig. 2

is from Lanza's series.* The apparent elongation was observed by extensometers in the Talbot series, and by direct readings in the Lanza series. The actual elongation was computed from the known value of the modulus of elasticity of the steel and from the value of

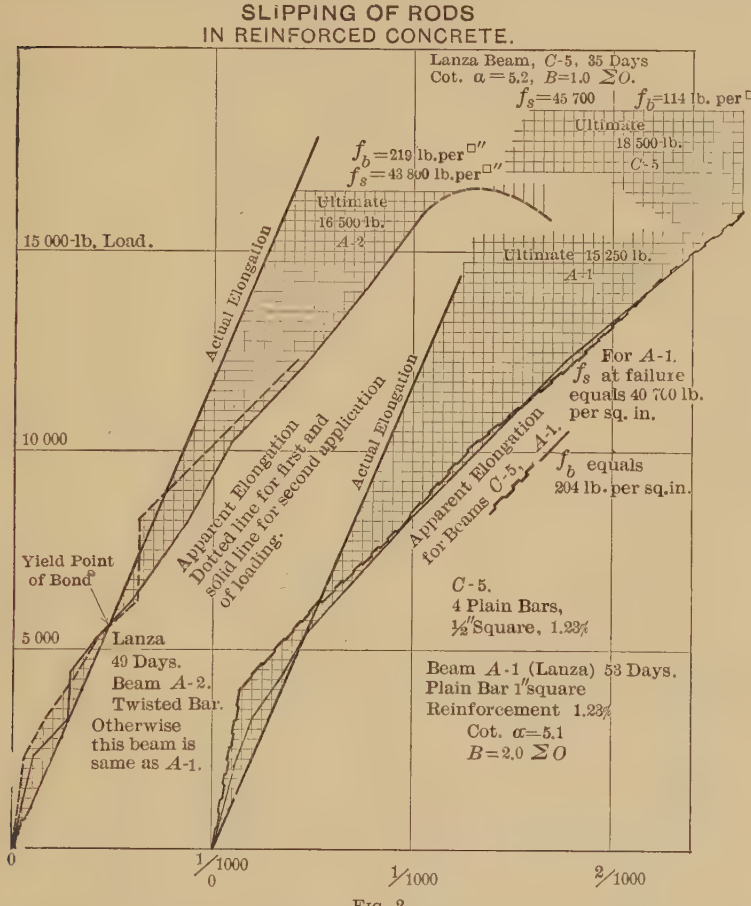


FIG. 2.

the tension produced therein, as calculated from the known forces acting on the specimen.

The Talbot beams, Nos. 27 and 28, were practically alike, with the exception that one was reinforced with plain bars and the other

* Reports on both these series may be found in *Proceedings*, Am. Soc. for Testing Materials, Vol. VI.

with deformed bars. The creeping seems to have begun on No. 28 (deformed-bar reinforcement) at about one-fourth of the ultimate load, and to have increased to nineteen-elevenths (1.72) of the actual elongation of the steel at the reading taken immediately before failure. This implies an apparent slip of 72% of the actual elongation. This point, where the creeping seems to have begun, may be called the yield point of the bond, and, for all practical purposes, may be taken as the point of intersection of the curves for actual and apparent elongation of the steel. On No. 27 (plain-bar reinforcement) the creeping seems to have begun at about two-thirds of the ultimate load, and the apparent elongation at the reading taken before failure was about 12 ÷ 10½ (1.14) of the actual. This implies an apparent slip of only 14% of the actual elongation.

The Lanza beams, *A*-1, *A*-2, and *C*-5, in Fig. 2, are very much like the Talbot beams, Nos. 27 and 28, as to make-up and quality, with a very slight variation as to age. On *A*-2 (deformed-bar reinforcement) the creeping seems to have begun at about one-third of the ultimate load, and the apparent elongation at the reading taken before failure was 22 ÷ 13½ (1.63) of the actual. This implies an apparent slip of 63% of the actual elongation. On *A*-1 (plain-bar reinforcement) the creeping seems to have begun at one-third of the ultimate load, and the apparent elongation at the reading taken before failure was 22½ ÷ 12 (1.88) of the actual. This implies an apparent slip equal to 88% of the actual elongation. On *C*-5 (plain-bar reinforcement) the creeping seems to have begun at about one-third of the ultimate load, and the apparent elongation at the reading taken before failure was 26½ ÷ 13¼ (2.0) of the actual. This implies an apparent slip equivalent to the actual elongation of the rod.

Before considering the Talbot beams, Nos. 16, 17, 19, and 21, it is desirable to define the exact meaning of "apparent elongation." The apparent elongation, which is observed on the side of the beam, is transmitted from the rod to the face of the concrete by the intervening concrete. It is fair to assume:

First.—That the strain transmitted to the intervening concrete by the rod must (when the bond is not broken) be a maximum on the fibers next to the surface of the rod, and zero at some point remote therefrom in the horizontal plane of the steel.

Second.—That that point may fall (*a*) on the surface of the beam, (*b*) between the surface of the beam and the surface of the rod, or (*c*) entirely outside of the surface of the beam. If the zero point falls within, then none of the strain (unless possibly of a negative nature) will reach the surface of the beam; on the other hand, if it falls without, then a certain proportion of the strain will reach the surface. In the above statements it will be recognized that the writer does not accept without question the historic assumption regarding plane sections in reinforced concrete beams.

Third.—Another circumstance that will also be apparent is the fact that, after the ultimate tensile strength of the concrete has been exceeded, the horizontal fibers of this material begin to tear apart and form irregular divisions along the tensile side of the beam.

It is conceivable that the bond may be preserved in some of these divisions, and more or less transformed to bond-friction-resistance in others. Some of the slip may be taken up in visible tension cracks and some by a number of microscopic cracks distributed in each of these divisions, but, as the load increases, these will tend to close and localize the elongation in a few definite fractures, more particularly those under the load or loads. The closing of the microscopic cracks will tend to hide most of the real elongation on that division if it should happen that readings are confined to that region. This is particularly true for the middle third of a beam with third-point load. That there is a true elongation in the steel, other than the apparent, may be inferred from the indications in the elongations for the Talbot beams, Nos. 16, 17, 19, and 21, and from the indications resulting (before creeping begins) from repeated loadings on the same beam, or from the first loadings on like beams. The apparent elongation was not even equal to the actual elongation when the maximum load was on beams Nos. 16, 17, 19, and 21. It was ten-elevenths for No. 17, and about the same for the others.

This is what occurred: The true elongation did not happen to appear on the surface within the region measured. It was in evidence, in an irregular way, outside of that region. The apparent elongation also for the second application of load for *A*-2 suggests that there is an elongation other than the apparent. This is evident when the apparent elongations for the first and second applications of load are compared, and is suggested by their relatively erratic behavior below the yield point.

The fact that, in cases of repeated loading, the rods would not always return to their original positions when the load was removed is made evident in the diagrams for deflection of beams tested with repeated loads by Professor Hatt.* This slipping and creeping of the reinforcement, although interesting and not without importance in itself, had only a secondary bearing on the analysis of the tests which follow. The same is the case with the question of plane sections before and after bending.

After several hundred beams had been analyzed, an attempt was made to divide them into groups, as to age and quality of concrete, but—other things being equal—it was observed:

I.—That, as the load on like beams approached the supports, the bond-friction-resistance increased;

II.—That, under no circumstance was there any indication of the high bond values that had been obtained from early experiments on bond by pulling rods from cubes of concrete;

III.—That those beams having a maximum of concrete between the rods gave very high results; and,

IV.—That those beams with a minimum of concrete between the rods gave very low results.

The first and second observations led to the belief that there must be two components in the bond-friction-resistance, a tangential and a normal component, and to the further beliefs that the normal component in the cases of pulling rods from cubes or cylinders of concrete was a compressive stress; that the normal component in beam tests was mostly tension, with more or less compression at and near the supports; and that this compression at and near the supports varied with the variations of the angle, α, shown in the diagram, Fig. 3. In the section of this diagram showing concrete below plane of steel, the values of the forces (7) and (9), are dependent on the values of (5). This section will also suggest the reason for the sense of the forces acting on the lower side of the rod. In this diagram will also be found further illustration of the distribution of the stresses just mentioned.

The reports, therefore, were all examined again and analyzed with

* *Proceedings*, Am. Soc. for Testing Materials; also published in *Engineering News*. October 24th, 1907.

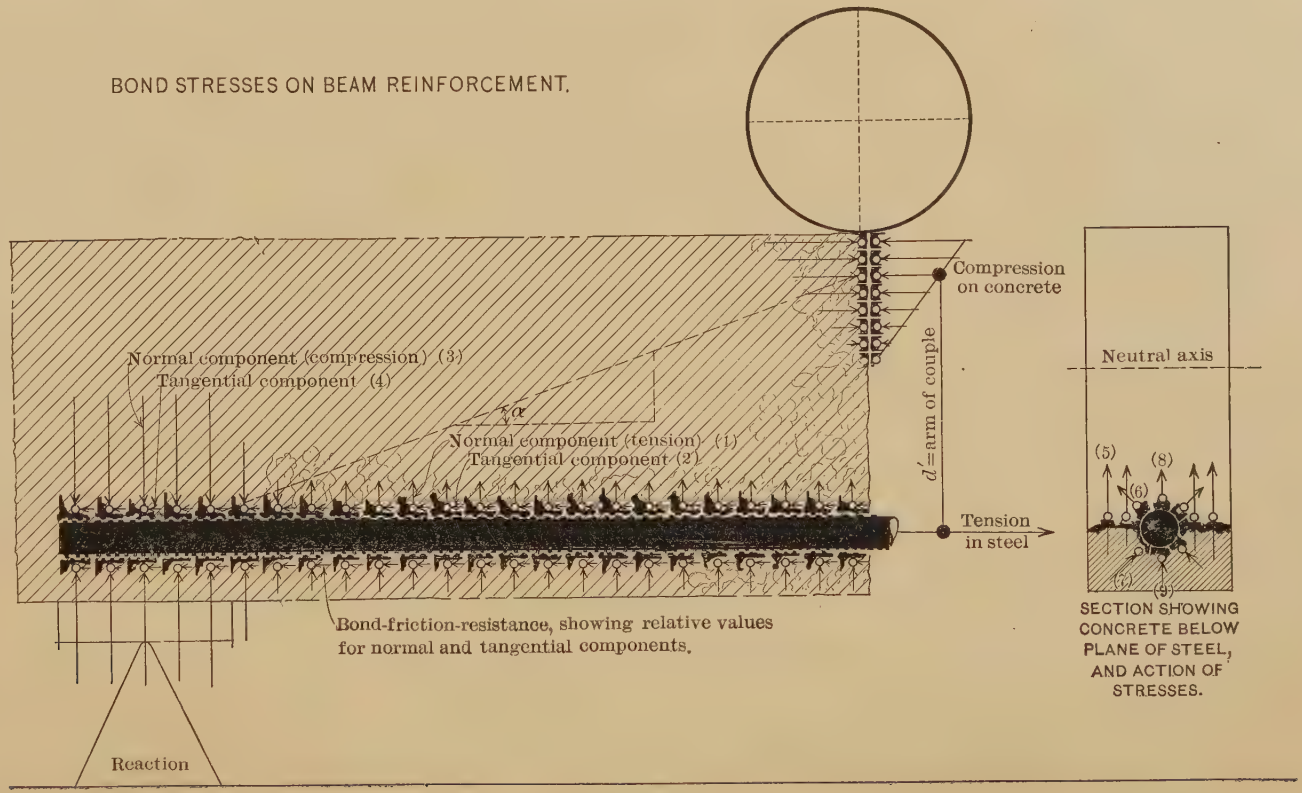

BOND STRESSES ON BEAM REINFORCEMENT.
Compression on concrete
Normal component (compression) (3)
Tangential component (4)
α
Normal component (tension) (1)
Tangential component (2)
d'=arm of couple
Tension in steel
Neutral axis
(5)
(8)
(6)
(7)
(9)
Bond-friction-resistance, showing relative values for normal and tangential components.
Reaction
SECTION SHOWING CONCRETE BELOW PLANE OF STEEL, AND ACTION OF STRESSES.

FIG. 3.

reference to these four elements. Groups were carefully sought which would have not more than one variable the influence of which it was desired to study. Although choosing from more than a thousand experiments, it is surprising how few of them it was possible to place in each group and keep free from foreign influences. However, the material was gradually gathered until sufficient information was obtained as to the influence of each element.

The first hint that the bond-friction-resistance increased as the loads approached the end supports of a beam was gained from a small group of experiments* in which the beams were identical in every way; dimensions of cross-section, length, span, reinforcement, depth of reinforcement, quality of concrete, and age at testing, were as nearly alike for all specimens as could be obtained in laboratory work. The only variable was the position of the load. The group was peculiarly fascinating to the writer, as it held some confusing contradictions. When the load was nearest the supports, the highest values for bond-friction-resistance were developed, yet when it was farthest from the supports, the highest values were developed for the tensile strength of the steel. This group did not seem to be sufficiently representative, but when Bulletin No. 14, from the same Experiment Station, came out with another group, identical in every way and giving results that agreed in every particular with it, the evidence seemed to be conclusive. However, further search was made, and quite an elaborate group was found in Bulletin No. 148 of the University of Wisconsin. The beams in this group were identical as to shape, size, span, and age; they were not as large as those tested in the University of Illinois, nor was the span as great. At the latter station the cement was measured by loose volume, but it was weighed at the University of Wisconsin, 100 lb. being rated as a cubic foot. The other materials were measured by loose volume at each station. The reinforcement in the first two groups was straight round rods without hooks, crooks, deformations, or web members. In the latter group the rods were also round, but they were bent up at various points. The influence of this bending, however, does not affect the comparative values resulting from the influence of position of load. The results from this group confirmed exactly those from the first two groups.

* Bulletin No. 4, University of Illinois, Engineering Experiment Station.

That is, there was high bond-friction-resistance when the load was near the supports and lower values as it moved toward the center of the beam. Evidently, the high values that result when the load is near the support are due to an increase in the value of the normal component of the bond-friction-resistance. For each group the length of the arm of the couple which forms the section moment on the cross-section under the load point is constant for every position of the load, therefore the only variable is the distance of the load from the support.

The ratio of these two lengths, therefore, will give an accurate value as a basis for determining the influence of this normal component as affected by the position of the load with reference to various dimensions of beams and various percentages of reinforcement. The variations of this ratio are represented by the cotangent of the angle, α, as abscissas on the diagrams, and the bond-friction-resistance as ordinates. The individual values for each beam and the mean values for like beams are both given for all three groups. The results are plotted on Figs. 4 and 5. It is believed by the writer that this bond-friction-resistance as given on these diagrams is only an average value, and that the true value varies at several points along the rod. At and near the supports it will be almost as great as is obtained from pulling a rod from a cube or cylinder of concrete, in which case the normal component is a compressive force. Outside the influence of the compressive forces acting on the beam, that is, within the influence of the tensile forces, it will be less than the average value given in these diagrams.

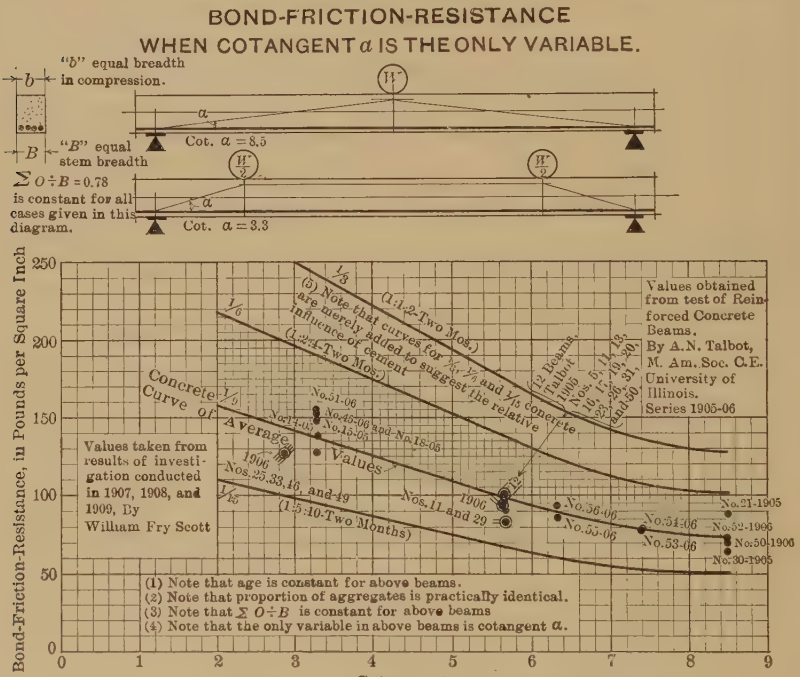
Bond-friction-resistance on steel with a high elastic limit, but stressed very much below it, will be higher, probably 3 or 4%, than on steel with a low elastic limit, but stressed a little beyond that limit —other things being equal.

As to the second observation, "that under no circumstance was there any indication of the high bond values that had been obtained from early experiments on bond by pulling rods from cubes or cylinders of concrete," the results of the most important tests on adhesion, as given by Turneaure and Maurer,* indicate that plain bars develop from 237 to 756 lb. per sq. in. for 1:2:4 concrete under such methods, while Fig. 13 indicates that plain bars develop from 50

* "Principles of Reinforced Concrete Construction."

to 110 lb. for 1:2:4 concrete when embedded in a reinforced concrete beam with a breadth of stem equal to two-thirds of the sum of their perimeters, as explained later.

Since this investigation was completed, there have been published* the results of two sets of experiments which are interesting as further confirmation of this second observation. The first is entitled "Tests on

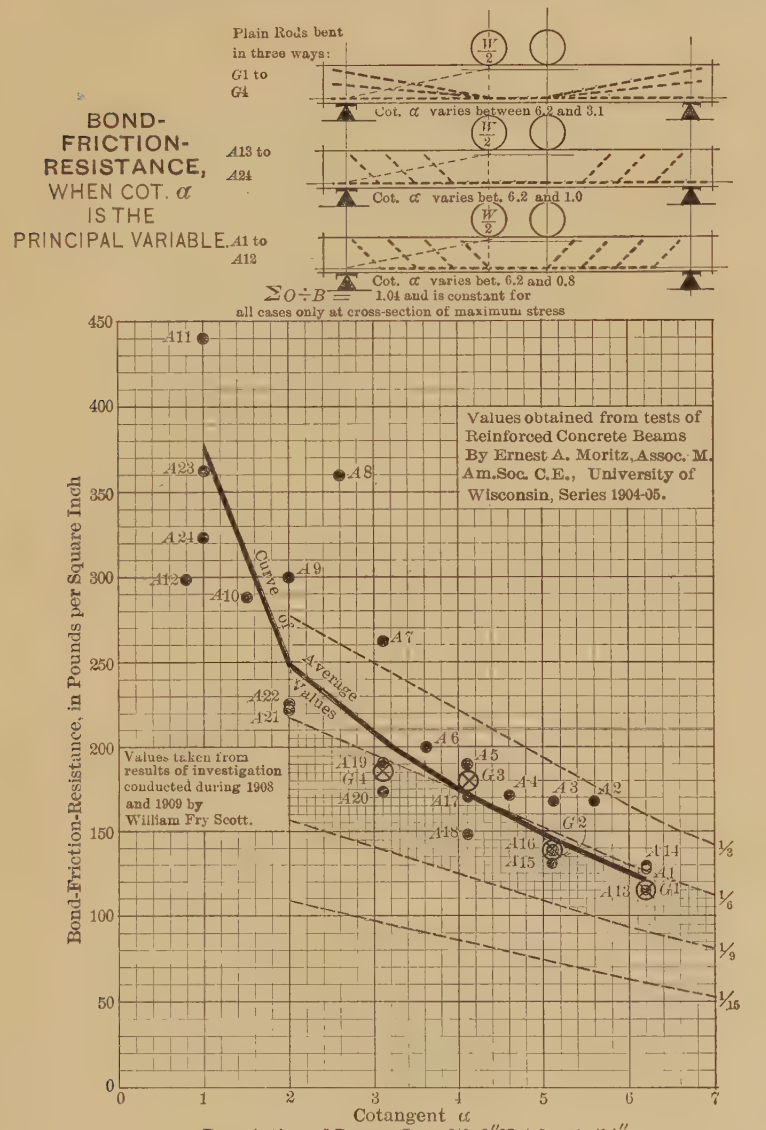

BOND-FRICTION-RESISTANCE
WHEN COTANGENT α IS THE ONLY VARIABLE.
"b" equal breadth in compression.
"B" equal stem breadth
Σ O÷B = 0.78 is constant for all cases given in this diagram.
Cot. α = 8.5
Cot. α = 3.3
Bond-Friction-Resistance, in Pounds per Square Inch
Cotangent α.
Values taken from results of investigation conducted in 1907, 1908, and 1909, By William Fry Scott
Values obtained from test of Reinforced Concrete Beams, By A.N. Talbot, M. Am. Soc. C.E. University of Illinois. Series 1905-06
(1) Note that age is constant for above beams.
(2) Note that proportion of aggregates is practically identical.
(3) Note that Σ O÷B is constant for above beams
(4) Note that the only variable in above beams is cotangent α.

Description of Beams: Net dimensions; Breadth 8," Depth 10,"
1905 Series 1:3:6 Concrete, 2 months old,
1906 Series 1:3:5½ Concrete, 2 months old.
Reinforcement: 4 Straight Rods, ½" ϕ.

FIG. 4.

Bond in Reinforced Concrete Beams," by Mr. Morton O. Withey, made in the Materials Testing Laboratory at the University of Wisconsin. The second is entitled "Some Tests of Bond of Steel Bars Embedded in Concrete by Three Methods," by Mr. H. C. Berry, made in the laboratory of the Civil Engineering Department of the University of Pennsylvania. The conclusions of Messrs. Withey and

* *Proceedings*, Am. Soc. for Testing Materials.


BOND-FRICTION-RESISTANCE, WHEN COT. α IS THE PRINCIPAL VARIABLE.
Plain Rods bent in three ways:
G1 to G4
A13 to A24
A1 to A12
W/2
Cot. α varies between 6.2 and 3.1
Cot. α varies bet. 6.2 and 1.0
Cot. α varies bet. 6.2 and 0.8
Σ O ÷ B = 1.04 and is constant for all cases only at cross-section of maximum stress
Values obtained from tests of Reinforced Concrete Beams By Ernest A. Moritz, Assoc. M. Am. Soc. C.E., University of Wisconsin, Series 1904-05.
Values taken from results of investigation conducted during 1908 and 1909 by William Fry Scott.
Curve of Average Values
Bond-Friction-Resistance, in Pounds per Square Inch
Cotangent α
1/3
1/6
1/9
1/15

Description of Beams: Breadth 3″, Net depth 4½″
Gross depth 5″, length 4′6″, Span 4′0″
Mortar 1:2½ (Note Cement was weighed, 100 lb. to the cubic foot which would make a mortar equivalent to 1:2 when measured by loose volume, the latter being the method used by Professor Talbot, Univ. of Illinois.)
Reinforcement: 4 Rods, ¼″ ϕ.

FIG. 5.

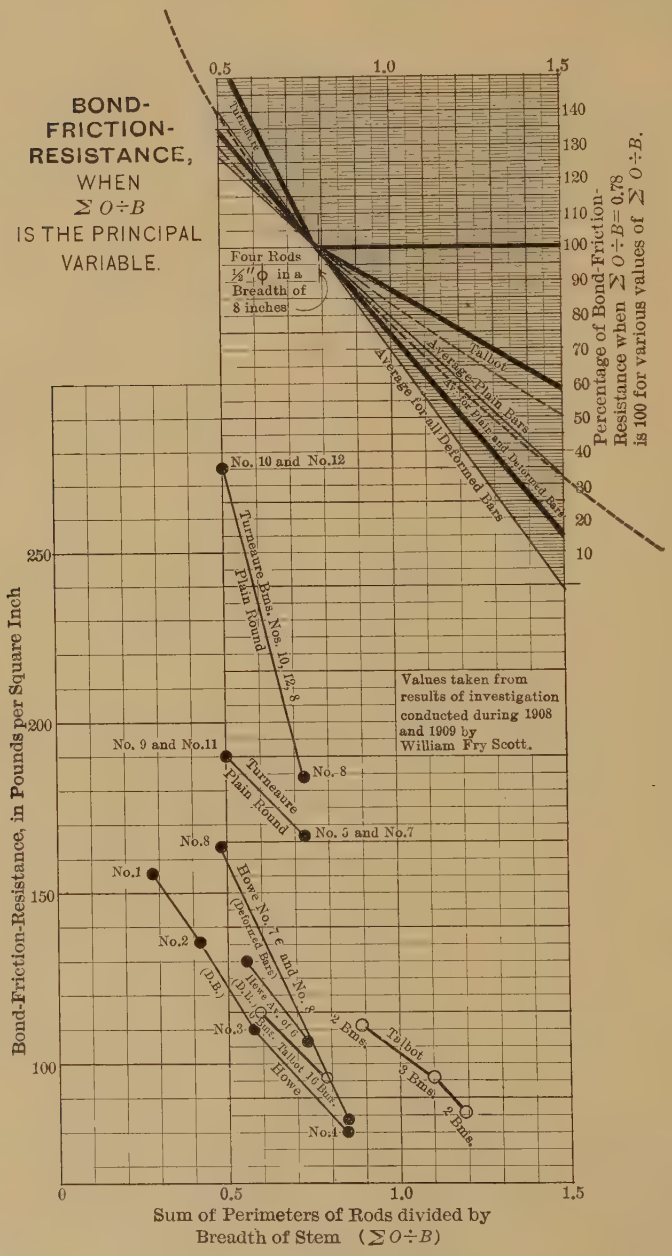

BOND-FRICTION-RESISTANCE, WHEN $\Sigma O \div B$ IS THE PRINCIPAL VARIABLE.
Percentage of Bond-Friction-Resistance when $\Sigma O \div B = 0.78$ is 100 for various values of $\Sigma O \div B$.
Four Rods ½" ϕ in a Breadth of 8 inches
Talbot
Average Plain Bars
Average for Plain and Deformed Bars
Average for all Deformed Bars
No. 10 and No.12
Turneaure Bms. Nos. 10, 12, 8
Plain Round
Values taken from results of investigation conducted during 1908 and 1909 by William Fry Scott.
No. 9 and No.11
No. 8
Turneaure
Plain Round
No. 5 and No.7
No.8
No.1
Howe No. 7 and No. 8
(Deformed Bars)
No.2
(D.B.)
Howe Av. of 6
(D.B.) 6 Bms.
No.3
Talbot 16 Bms.
Howe
No.4
2 Bms.
Talbot
3 Bms.
2 Bms.
Bond-Friction-Resistance, in Pounds per Square Inch
Sum of Perimeters of Rods divided by Breadth of Stem ($\Sigma O \div B$)

FIG. 6.

Berry are similar, and therefore it will be sufficient to quote the latter who found that "push-out" tests gave two and one-tenth times, and "pull-out" tests two and six-tenths times, the bond stress that had been obtained on their beam tests. He says, of these experiments:

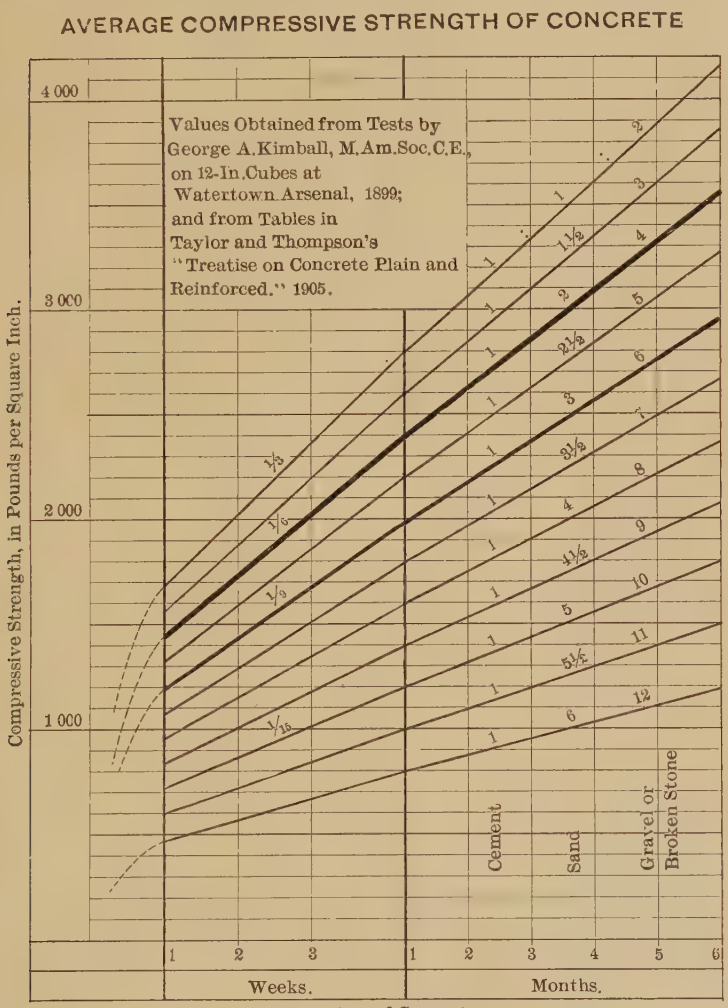


FIG. 7.

"It is evident * * * that the maximum value of the bond between concrete and plain steel in reinforced concrete beams is between 200 and 250 lb. per sq. in., and that this value instead of the higher values usually quoted should be taken for guidance in reinforced concrete design."

It will be noted that these results are the same as the minimum value obtained from "pull-out" tests quoted from Turneaure and Maurer. This is easily explained. The specimens in the Withey tests had a value of 5 for cot. α, and those in the Berry test a value of 2; and in both cases the breadth of the concrete was two and one-half times the sum of the perimeters of the reinforcing rods. Therefore, according to Fig. 13, the Withey tests should have given more than 200 lb. per sq. in., and the Berry tests nearly 400 lb. per sq. in., if the rods in these specimens had extended into the supports.

That they did not extend into the supports, and that one-half of the average bond-friction-resistance was all that was developed on the rods, is convincing proof that the writer's theory—that the bond-friction-resistance varies at different points along the rod—is true. The Berry rods extended 8 in. only, in each direction from the center of the beam, that is, one-third of the distance to the support, thus they were entirely within the field of the tensile stresses in the concrete, instead of reaching into the field of the compressive stresses at the supports.

With regard to the confusing contradictions previously mentioned as existing in the two groups illustrated in Figs. 4 and 5—that is, when the load was nearest the supports the highest values for bond-friction-resistance were obtained, yet when the load was farthest from the supports the highest values were obtained for the tensile strength of the steel—the writer has only recently arrived at a positive explanation of this anomaly. The theory underlying the explanation had been developed some time ago, having been held in abeyance until an opportunity should arise to prove it. The secret may be found from two facts:

1.—The maximum tensile stress in a rod embedded in a beam that is centrally loaded is localized, rather then distributed over the entire length of the rod; thus a local increment of elongation reaching 7 or 8% will not appreciably affect the total elongation over the whole length of the rod. The exact distribution of the elongation is represented graphically in Figs. 8 and 9 for center point and one-third point load.

2.—At 90% of the ultimate strength of the steel the elongation does not exceed 6 or 7% when the rod is uniformly stressed from end

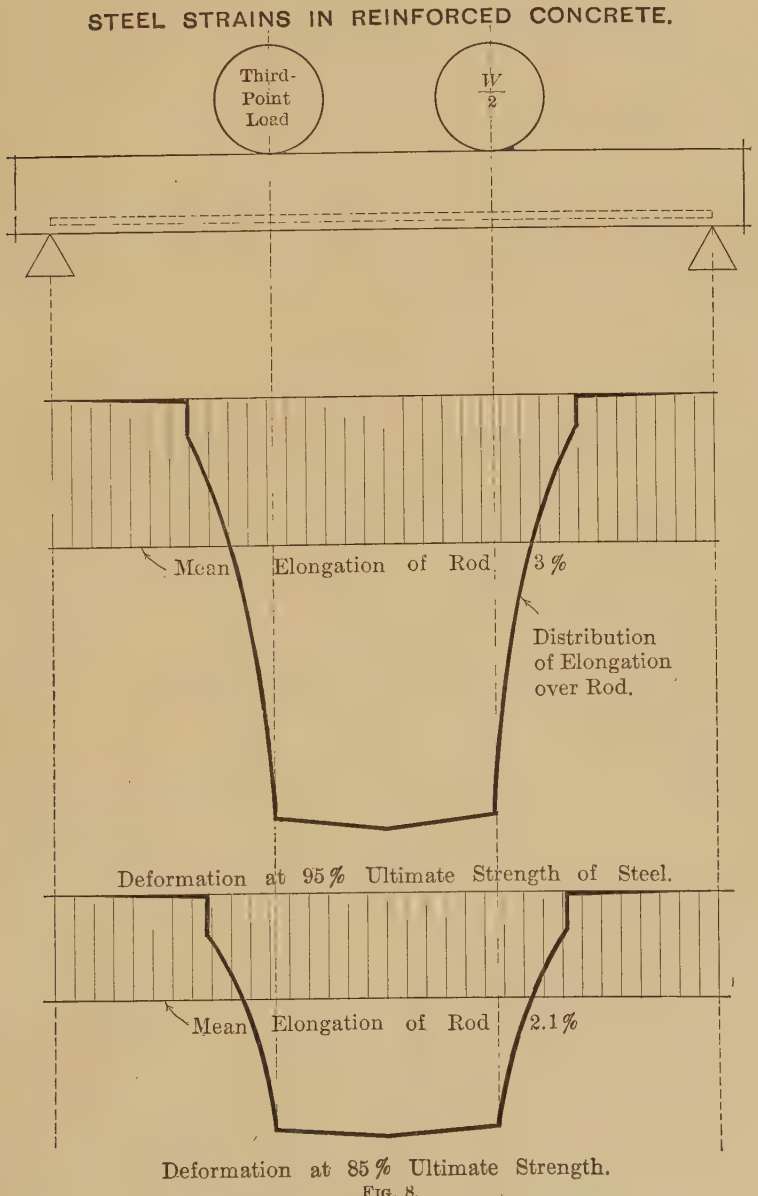

STEEL STRAINS IN REINFORCED CONCRETE.
Third-Point Load
W/2
Mean Elongation of Rod 3%
Distribution of Elongation over Rod.
Deformation at 95% Ultimate Strength of Steel.
Mean Elongation of Rod 2.1%
Deformation at 85% Ultimate Strength.

FIG. 8.

to end; at 80%, from 3 to 4%, and at 70%, say, 2% elongation. (See Figs. 10, 11, and 12.)

From this evidence it will be seen that the region of plasticity of steel and iron, between the yield point and from 85 to 95% of the ultimate strength, has an important influence in reinforced concrete. The writer is of the opinion that rods thus tested will give a higher average of values than is obtained from ordinary tension tests, the reason for which is the fact that market steel is not absolutely uniform, and that the point where the maximum stress is forced to become localized is not likely to coincide with the weakest point in the material, but will more probably occur where the material has an average value.

This will be understood from a study of Figs. 10 and 11. These give the results of tests on $\frac{7}{8}$-in. rods, 6 ft. long, with a measured length of 48 in., or six standard measured units. Measurements were taken from period to period as the load progressed. The readings were taken for each inch over the entire measured length. The tests, made in the Engineering Laboratory of the University of Toronto on a 100-ton Riehlé machine, were undertaken for the express purpose of obtaining information for the construction of the two diagrams, Figs. 8 and 9. The writer was assisted in these tests by Mr. Peter Gillespie, Lecturer in Applied Mechanics in the University of Toronto, Mr. Kenneth Alexander MacKenzie, and Mr. Herman Hyman, all of whom he wishes to thank for the able manner in which the work was conducted. Special gratitude is due to Mr. Gillespie for the interest shown by him in this problem. The results of these tests are further supplemented by the diagram, Fig. 12, which gives a carefully prepared curve for the elongation of steel and iron within the region of plasticity. The abscissas represent the percentages of elongation and the ordinates the percentages of the ultimate load. This diagram is presented for the purpose of indicating the wide variation in the location of the yield point. The variations in the elongation were less variable, and seemed to fall indifferently on either side of the curve for both common iron and hard steel.

Regarding the third and fourth observations—as to the influence of the concrete between the rods in the plane of the reinforcement—it is commonly assumed that a breadth of one diameter of clear concrete between the rods is ample to develop the bond strength of the

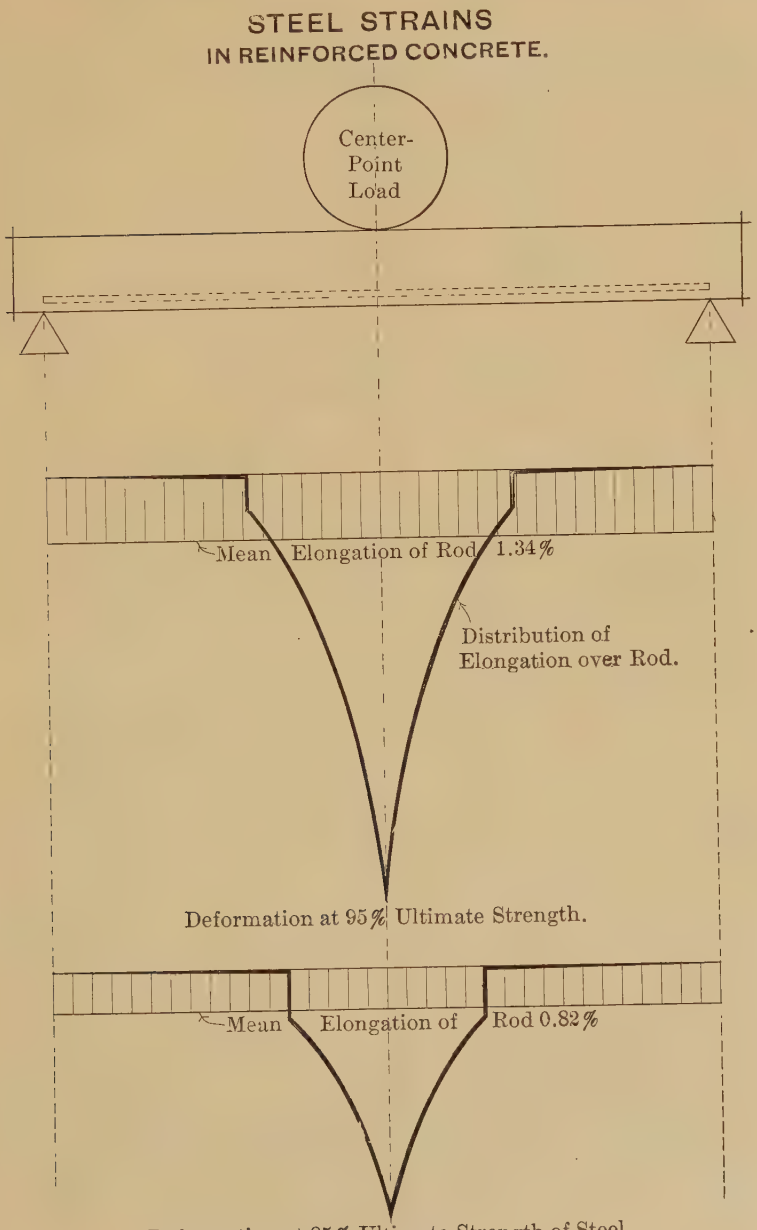

STEEL STRAINS
IN REINFORCED CONCRETE.
Center-
Point
Load
Mean Elongation of Rod 1.34%
Distribution of
Elongation over Rod.
Deformation at 95% Ultimate Strength.
Mean Elongation of Rod 0.82%
Deformation at 85% Ultimate Strength of Steel.

FIG. 9.

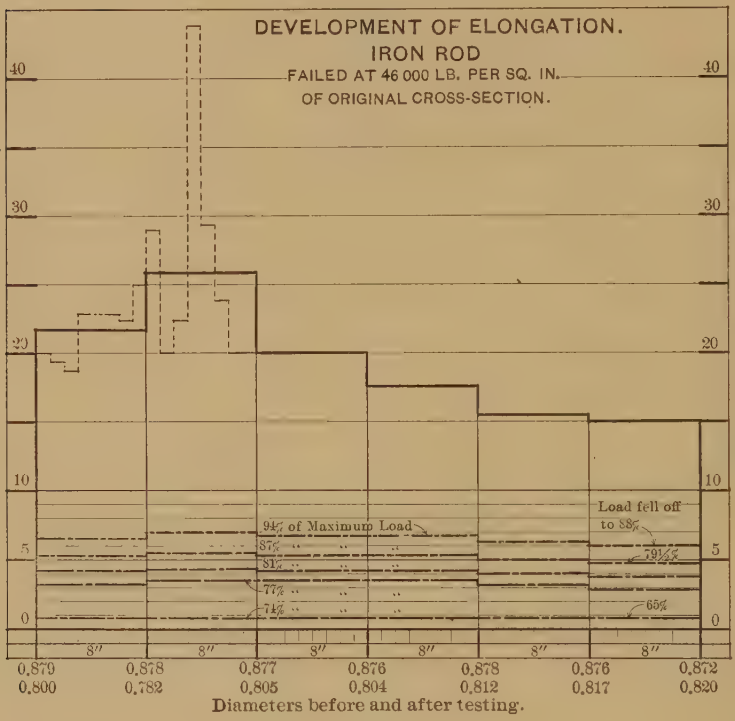

DEVELOPMENT OF ELONGATION.
IRON ROD
FAILED AT 46 000 LB. PER SQ. IN.
OF ORIGINAL CROSS-SECTION.
Percentage of Elongation.
94% of Maximum Load
87% " " "
81% " " "
77% " " "
74% " " "
Load fell off to 88%
79½%
65%
8″ 8″ 8″ 8″ 8″ 8″
0.879 0.878 0.877 0.876 0.878 0.876 0.872
0.800 0.782 0.805 0.804 0.812 0.817 0.820
Diameters before and after testing.

FIG. 10.

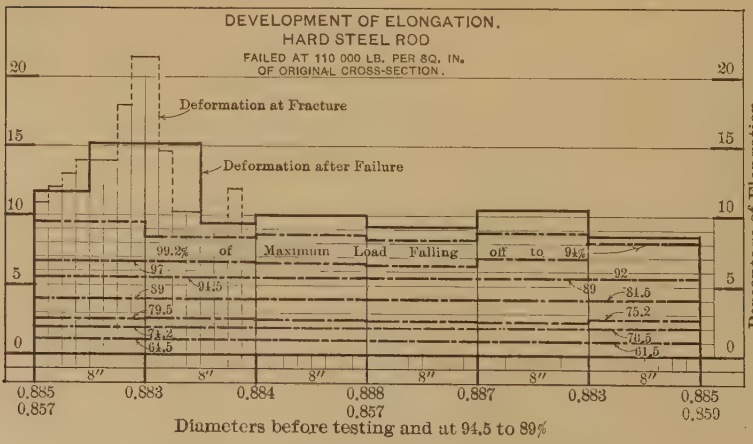

DEVELOPMENT OF ELONGATION.
HARD STEEL ROD
FAILED AT 110 000 LB. PER SQ. IN.
OF ORIGINAL CROSS-SECTION.
Deformation at Fracture
Deformation after Failure
Percentage of Elongation.
99.2% of Maximum Load Falling off to 94½
97
94.5
89
79.5
74.2
91.5
92
89
84.5
75.2
78.5
81.5
8″ 8″ 8″ 8″ 8″ 8″
0.885 0.883 0.884 0.888 0.887 0.883 0.885
0.857 0.857 0.859
Diameters before testing and at 94.5 to 89%

FIG. 11.

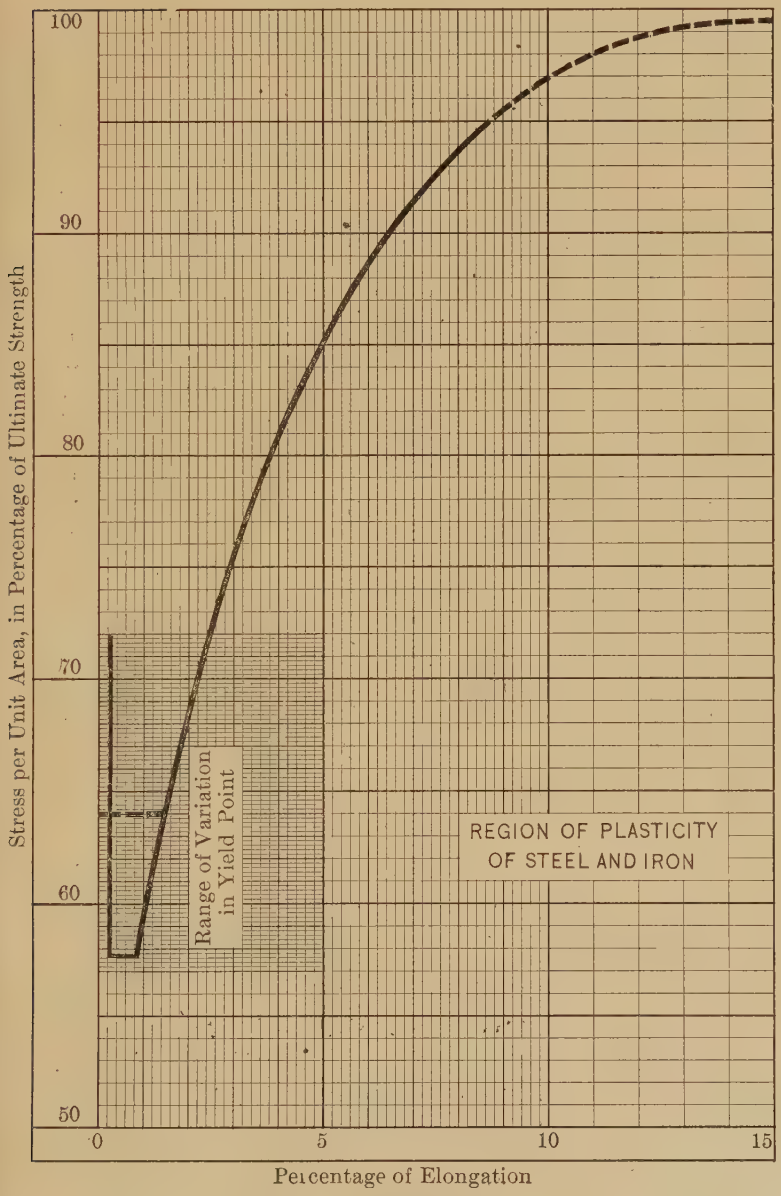

100
90
80
70
60
50
0
5
10
15
Stress per Unit Area, in Percentage of Ultimate Strength
Percentage of Elongation
Range of Variation in Yield Point
REGION OF PLASTICITY OF STEEL AND IRON

FIG. 12.

lower half surface of round rods, or of square rods when their diagonals are in the plane of the rods. The following, which is quoted from Turneaure and Maurer, fairly represents the generally accepted theory with regard to the spacing of rods or bars:

"In rectangular or T-beams the spacing of bars is important; in T-beams this consideration will largely control the width of the beam. The requirement in general as to spacing is that the amount of concrete left between the bars must be sufficient to transmit to the upper part of the beam the stress which the bars give over to the concrete below them. If the bars are circular it may be assumed that one-half of the stress in them is given over to the concrete below, hence the strength of the concrete on a longitudinal section through the center plane of the bars must equal one-half of the stress in the bars. If the shearing stress be taken as equal only to the bond stress then the clear space between bars must be one-half the circumference of a bar, or 1.57 diameters. In the sense here employed the shearing strength is at least twice the bond strength for smooth rods, so a clear spacing of less than one diameter is sufficient from this standpoint. In the case of square bars, on the same basis, the clear spacing would need to be 1½ diameters if the bars are placed with sides vertical, or one diameter if placed with sides diagonal. But in addition to the shearing stresses there is likely to be developed more or less tension in the concrete surrounding the rods, so that there should be left ample areas of concrete between them, especially towards the end where the bond stresses are large. * * * A minimum clear spacing of at least 1½ diameters should be provided, with an equal distance between the outside rod and the surface of the beam. Where some of the rods are bent up the spacing can readily be made more liberal towards the end of the beam."

In conformity with this theory, the proposed specifications of the Special Committee on Concrete and Reinforced Concrete require that, "The lateral spacing of parallel bars should not be less than two and one-half diameters, center to center." The extra half diameter is provided for the purpose of allowing for the possibility of the placing of square bars with the sides diagonal. Owing to the great variety of forms of cross-sections of bars, it is believed by the writer that the relation between perimeter and breadth will establish a better basis for determining the correct values for bond-friction-resistance than the relation between diameter and breadth, because a bar with a major and minor axis has no fixed diameter.

According to the theory outlined by Turneaure and Maurer, the

breadth of the concrete should never be less than two-thirds of the sum of the perimeters of the bars, but that is assumed to be ample.

The question was investigated carefully by the writer, and, though the data were rather meager, sufficient were gathered to prove conclusively that, say, three times the above-mentioned allowance for breadth of concrete will develop more than two and one-half times as much bond-friction-resistance.

The first hint that the writer received that the breadth of concrete between the rods was of more vital importance than was commonly supposed was obtained from a series of tests of beams by Malverd A. Howe, M. Am. Soc. C. E., of Rose Polytechnic Institute, in 1903.* The Howe series consisted of 18 beams, the reinforcement was old-style corrugated bars, and the ages were from 10 to 11 weeks. The concrete was composed of 1 of cement to 7 of aggregates. The percentage of reinforcement was practically 0.7 of 1% of the net cross-section of the concrete for all the beams. No two beams had like dimensions. Beams Nos. 1, 2, 3, and 4, were almost identical, with regard to the possible modifying influences, except that of the ratio of the perimeter of the rods to the breadth of the concrete in the plane of the reinforcement. A very decided rate of increase was noted in the resulting bond-friction-resistance developed on these four beams due to the increase in breadth of concrete between rods. Besides this series just mentioned, several other groups were found for both plain rods and deformed bars. These groups have been plotted on Fig. 6, the ordinates giving the bond-friction-resistance in pounds per square inch of the surface of the bar, and the abscissas giving the ratio of the perimeter of the rods to the breadth of the beam. In each individual group of beams every beam had a common value for cot. α, thus making the normal component of bond stress constant for that group.

The following method was devised for establishing a relation between the values on this diagram and the values previously determined in Fig. 4: In the beams represented in Fig. 4 the sum of the perimeters of the rods was equal to $\frac{7.8}{100}$ of the breadth of the concrete, therefore this value for the spacing of the rods was adopted as the unit value for comparison of the results in Fig. 6. A point was chosen on the ordinate passing through the abscissa ($\frac{7.8}{100}$), but near the top of

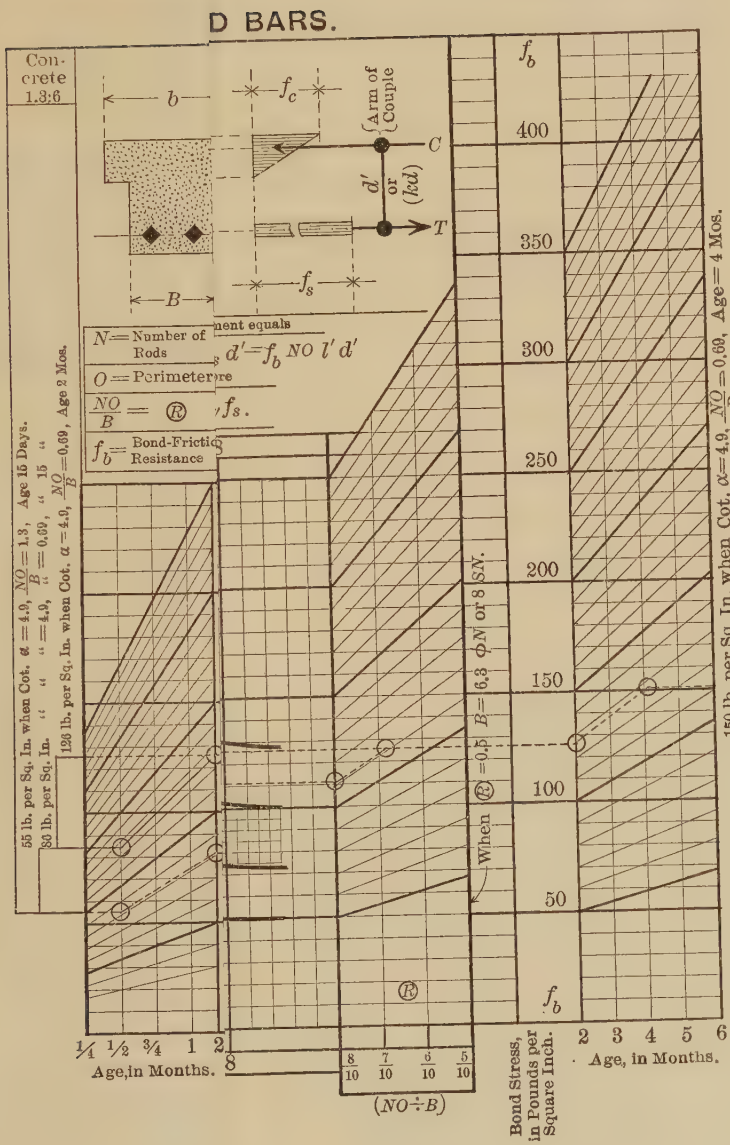
* A Report of these tests may be found in the *Proceedings*. Am. Soc. for Testing Materials, in Mr. Condron's discussion of Professor Talbot's tests, Series of 1904.

the diagram. This point then became the nucleus of a new set of diagrammatic values with the abscissas as before and the ordinates to be established in the following manner:

Through this nucleus, lines parallel to those in the lower figure were drawn. Where an average direction for two or more groups was desired, the protractor was used to obtain the mean inclination of the series with respect to the horizontal axis. Thus a line representing the entire series of plain bars was constructed; similarly, curves for the entire series of deformed bars, and for all the bars, deformed and plain, were obtained. These are the values to be found on the upper diagram in Fig. 6. It will be noticed that there is practically no difference in the results obtained for plain or for deformed bars—both seem to be equally influenced by the increase or decrease of the concrete between the bars.

The specimens in the preceding investigation were practically two months old, therefore the values are only true for that age. The writer found no sufficiently elaborate reports of tests on beams to enable him to establish a basis for judging of the influence of age of concrete on bond-friction-resistance. Therefore, a careful study was made of the investigations of Kimball, as conducted at the Watertown Arsenal, and of the theories of Taylor and Thompson, as given in "Concrete, Plain and Reinforced," on the effect of age on the compressive strength of concrete. The results of this study are given on Fig. 7. It is fair to assume that the influence of age on concrete is relatively the same, whether it be stressed in compression, in tension, or in bond. This assumption was made for the values as plotted on Plate XV, and odd experiments on age influence seem to bear out the probable truth of the assumption.

Further explanation as to the method of construction of the additional curves on Fig. 4 is necessary here. It will be recalled that the curve for the 1:9 mixture was plotted from a very large number of experiments, the individual values being shown adjacent to the curve. Talbot's experiments, Series 1907-08, provided the information which enabled the writer to construct the curves for the 1:15, the 1:6, and the 1:3 mixtures, as shown. These experiments show that for a given value of cot. α (2.8 in this specific instance), the values of the bond-friction-resistance—other things being equal—were as follows:


D BARS.
Concrete 1:3:6
b
f_c
Arm of Couple
C
d' or (kd)
T
f_s
B
N = Number of Rods
O = Perimeter
$\frac{NO}{B}$ = ®
f_b = Bond-Friction Resistance
$d' = f_b\, NO\, l'\, d'$
f_s.
55 lb. per Sq. In. when Cot. $\alpha = 4.9$, $\frac{NO}{B} = 1.3$, Age 15 Days.
80 lb. per Sq. In. " " $= 4.9$, " $= 0.69$, " 15 "
120 lb. per Sq. In. when Cot. $\alpha = 4.9$, $\frac{NO}{B} = 0.69$, Age 2 Mos.
150 lb. per Sq. In. when Cot. $\alpha = 4.9$, $\frac{NO}{B} = 0.69$, Age = 4 Mos.
Concrete 1:3:6.
When ® = 0.5 B = 6.3 ΦN or 8 SN.
f_b
400
350
300
250
200
150
100
50
1/4 1/2 3/4 1 2
Age, in Months.
8
8/10 7/10 6/10 5/10
(NO ÷ B)
2 3 4 5 6
Age, in Months.
Bond Stress, in Pounds per Square Inch.

For a 1:9 mixture.....100 (assumed)
" " 1:6 " 138
" " 1:3 " 176
" " 1:15 " 68

It was then assumed that for other values of cot. α, the same relations would obtain. Hence the 1:6 curve was drawn so that each of its ordinates is 38% greater than the ordinate on the 1:9 curve; for like values of cot. α; similarly, all values of the ordinates to the 1:15 curve are 68% of the values of the ordinates to the 1:9 curve. A glance at Fig. 7 indicates the relative compressive strength of various grades of concrete at one month to be:

For a 1:9 mixture.....100 (assumed)
" " 1:6 " 120
" " 1:3 " 140
" " 1:15 " 60

The writer believes that the bond-friction-resistance is affected to some extent by richness of mixture, in a manner similar to that in which compressive strength is modified by the same influence. He has the additional conviction that the curves shown on Fig. 4 are probably somewhat high, and that values of the ordinates somewhere in the neighborhood of the mean of those just quoted for compressive strength, and those from which the curves were plotted would more nearly represent the facts. He recognizes also that variation in material, manipulation, and method are quite unavoidable, under all circumstances; that in any kind of experimental investigation a close degree of refinement is absolutely impossible, and that the unbiased investigator must never lose sight of the personal equation in individual experiments.

Plate XV is a composite diagram, constructed from the results in Figs. 4, 6, and 7. Therefore it gives a comprehensive picture of the whole investigation, and will serve for laboratory and investigation purposes as a means for checking the accuracy of these new laws. It is rather too complicated for daily office use in reinforced concrete design, for which purpose Fig. 13 has been prepared. The latter is a double diagram, for concrete 1 month old and mixed in the proportion of 1 of cement to 2 of sand and 4 of coarse aggregates. The abscissas

in the left-hand diagram give values for cot. α, and the ordinates the bond-friction-resistance. The abscissas in the right-hand diagram give values for the breadth of concrete divided by the sum of the perimeters of the rods, and the ordinates are multiples of the ordinates which must first be obtained from the left-hand diagram. It will be noted that the values in this diagram are made for the standard concrete for use in reinforced concrete as laid down by the Special Committee on Concrete and Reinforced Concrete.

What has gone before refers to reinforcement with plain bars, embedded in the bottom of the concrete beam, with their axes parallel to its top. This was adopted as a type, for comparison with alternative or modified methods as alleged improvements. Improvement as here considered is any change made in this type. For instance, the following are some of the changes which have been considered in this class:

a. Deformations on rods,
b. Hooks on the ends of rods,
c. Bending rods upward to near the top of beam,
d. Stirrups for the web of the beam,
e. Combinations of two or more of the preceding.

Special care was taken to investigate thoroughly the standard type of reinforcement, so as to establish values with which to compare other alleged improvements. As a matter of fact, there were more data for the standard than for the others; however, sufficient were gathered to warrant the following statements:

a.—Deformations as media for increasing bond strength are of doubtful value. Any increase in strength resulting from the use of deformed bars is found to show itself only after the yield point of the bond has been passed. It will be noted in the studies for steel elongation in reinforced concrete beams that deformed bars sometimes exhibit a slower development in the slipping than is indicated by plain bars. A good illustration of this will be found by comparing Lanza beam *A*-2 with the Lanza beams *A*-1 and *C*-5, in Fig. 2. It will also be noted that deformed bars sometimes begin to creep earlier than plain bars. This is indicated by comparing the Talbot beams Nos. 27 and 28 in Fig. 1, and it will be found that this is by no means an uncommon occurrence; it may be due to a difficulty in getting the concrete to reach into the corners of the deformations.

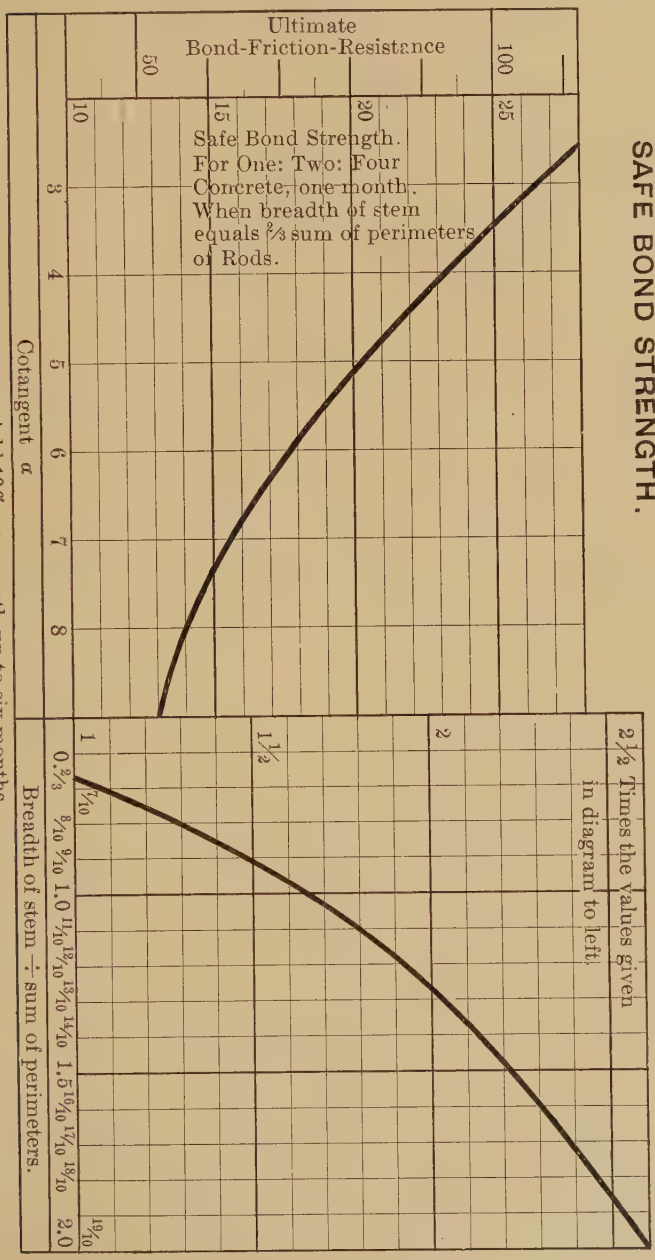

SAFE BOND STRENGTH.
Ultimate
Bond-Friction-Resistance
50
100
10
15
20
25
Safe Bond Strength.
For One: Two: Four
Concrete, one month.
When breadth of stem
equals ⅔ sum of perimeters
of Rods.
2½ Times the values given
in diagram to left.
2
1½
1
⅔
Cotangent α
3
4
5
6
7
8
Breadth of stem ÷ sum of perimeters.
7/10
8/10
9/10
1.0
11/10
12/10
13/10
14/10
1.5
16/10
17/10
18/10
19/10
2.0
Add 10% per month up to six months

Fig. 13.

The writer is of the opinion that, with proper tamping of concrete, the deformed bars of the twisted and corrugated type would probably give about 15 or 20% increase over plain bars.

The re-rolling necessitated in the manufacture of deformed bars usually has the effect of raising the elastic limit of the steel, and it is to the increase thus produced that a great deal of the merit of deformed over plain-bar reinforcement may be attributed. In the majority of cases it will be found that the bond-friction-resistance developed on deformed bars seldom exceeds, in fact seldom seems even to reach, that developed on plain bars under similar conditions. Now and again a group of beams will be found to indicate 30 or 40% increase due to the deformed bars. This seems to be particularly the case when the beams have been cured in water. This would cause a tightening of the grip of the concrete on the deformations of the rods resulting from expansion. Such results, however, have not been general, and, further, they have not been found to vary greatly from those given for the standard reinforcement on Plate XV; or, in other words, it has usually been the case that the plain bars with which they were compared behaved very badly. In every case average values for any of those groups varied less from the diagrammatic values than the variations in like beams in any group. This was true when the groups were from different laboratories, in which case the averages would also vary according to the practices of each laboratory. Thus the variations in the values in such groups may be considered as due mainly to the erratic nature of the concrete rather than to the influence of the deformations on the bars. Witness the series of groups in Table N-1, Bulletin No. 148, of the University of Wisconsin, by Ernest A. Moritz, Assoc. M. Am. Soc. C. E. In no case are there developed as high values for bond-friction-resistance for deformed bars as for plain bars.

This series may be assumed to be a representative one because of the fact that there are never less than four beams (usually five) for each kind of reinforcement, and nearly the same for each age, 1, 3, and 6 months, as shown on Table 1. In every case there is a much greater variation between the average values and the maximum and minimum values for each group than between the average values of the several groups and the values to be obtained from Plate XV. The minimum variation from the averages for each group was in one case 54% and the maximum variation 147 per cent. The minimum

TABLE 1.—Deformed versus Plain Bars. Moritz Tests on Reinforced Concrete Beams. All Beams of 1 : 3 Mortar. Each Beam Contains One Steel Rod.

Age, in months.	Plain Bars.		Deformed Bars.	
	½-in. round.	⅜-in. round.	Twisted, ½-in. square.	Corrugated, ½-in.
	A_s = 0.197 sq. in. $\frac{B}{\Sigma O}$ = 1.93 Cot. α = 4.5	A_s = 0.110 sq. in. $\frac{B}{\Sigma O}$ = 2.55 Cot. α = 4.35	A_s = 0.25 sq. in. $\frac{B}{\Sigma O}$ = 1.5 Cot. α = 4.55	A_s = 0.18 sq. in. $\frac{B}{\Sigma O}$ = 1.77 Cot. α = 4.5
	Loads at Failure.			
One.	4 000, 5 250, 4 100, 4 500, 4 400, 4 500 | Av. = 4 460 | f_b = 292	2 600, 3 000, 2 700, 2 000 | Av. = 2 575 | f_b = 215		
Three.	2 800, 4 200, 6 000, 5 600, 6 500 | Av. = 5 020 | f_b = 328		5 500, 5 000, 4 600, 4 600, 5 800 | Av. = 5 100 | f_b = 263	4 400, 4 600, 4 650, 5 500, 5 500 | Av. = 4 930 | f_b = 296
Six.	5 300, 4 900, 5 600, 4 600 | Av. = 5 100 | f_b = 333		6 000, 2 200, 2 640, 5 660 | Av. = 4 075 | f_b = 211	6 200, 5 000, 6 000, 5 400, 4 200 | Av. = 5 360 | f_b = 322

variation of the averages for each group is in one case 62% of the values in the diagram, and, strange to say, that was for a group of deformed bars. Considering only the plain-bar reinforcement in this series, the minimum variation of these experimental values from those on the diagram is 81% and the maximum 130% of the diagrammatic values. Also, in the Lanza and Talbot beams, there cannot be said to be any appreciable increase in the ultimate bond-friction-resistance due to the deformations on bars. In the case of Talbot beam No. 9, which was reinforced with three twisted bars, ½ in. square, the ultimate bond-friction-resistance was 168 lb. per sq. in., while in Nos. 16 and 17, which were each reinforced with three plain bars, ½ in. square, the ultimate bond-friction-resistance was 81 and 82 lb. per sq. in. In both cases $B = 2\,\Sigma O$ and cot. $\alpha = 5.2$. Now, according to Plate XV,

the average value for bond-friction-resistance for plain-bar reinforcement, as in Nos. 16 and 17, is 145 lb. per sq. in. Thus it is evident that these two beams must have been of very poor concrete; on the other hand, beam No. 9, with deformed-bar reinforcement, developed only 16% more than the average value obtained where plain bars were used. Beam No. 13 developed 104 lb. per sq. in., or 16% more than the average; No. 28 developed 105 lb. per sq. in., or 40% more than the average; while Nos. 3 and 7 developed only 67% of the average value for the bond-friction-resistance given on the diagram. The foregoing are the deformed bars. The plain bars behaved in a similar manner. Beam No. 21 developed 60%; No. 19, 60%; and No. 27, 110% of the average values on Plate XV. In other words, it is apparent that variations in bond-friction-resistance are attributable to such influences as age, the position of the load, the ratio of steel perimeter to width of stem, etc., rather than to special deformations in the reinforcement, except as indicated above. Yet, if the writer's suspicions regarding the values for the 1:6 concrete in Plate XV and Fig. 13 are correct, that is, that the ordinates may be, say, 7% too high, the foregoing deductions or comparisons are subject to a slight modification in favor of the deformed-bar reinforcement.

b.—Hooks on the ends of rods do not produce any increase in the strength of the reinforcement. They even tend to cause failure to occur sooner than when they are omitted.

With regard to the combination of hooks, bends, and stirrups in the reinforcement of a concrete beam, a rather curious anomaly is apparent in the results of any experiments on such a combination of which the writer has been able to obtain reports. In no case does such a combination show a materially higher development of bond-friction-resistance than the plain straight-rod reinforcement without hooks, bends, or stirrups, yet the effect of bending some rods without the use of stirrups or hooks, materially increases the bond-friction-resistance, and, also, when stirrups alone are added to the plain straight-rod reinforcement, a marked increase occurs. The separate influence of these will now be considered.

c.—Beams with reinforcing bars bent up have been investigated by Talbot. The following is from the results of his investigation.*

* Bulletin No. 29, University of Illinois Engineering Experiment Station.

"As may be expected, in the beams with all the bars bent up, as soon as the concrete at the bottom was sufficiently stretched, a vertical tension crack formed at some point between the beam support and the bend of the bar, extending from the bottom of the beam to the level of the reinforcing bars. * * * After the formation of this vertical crack, the tension in this part of the beam is taken mainly by the reinforcing bar; and as its depth below the top of the beam * * * is considerably less than in beams having the bars horizontal throughout, the tensile stress developed in the bar at this section is greater than in beams having the reinforcement horizontal throughout. * * * As the tensile stress in the steel at this section is greater than when the bars are horizontal, the amount of bond brought into action toward the end of the bar must be greatly increased. There is evidently, then, a greater chance for slip of bar in such beams than in beams where straight reinforcement is used."

Comparing the results of beams with all the bars bent up and the results of beams with all straight bars, there was no material difference in the bond-friction-resistance when the stresses on the cross-section under the load were considered for both cases. The writer believes that, on one hand, there is a gain due to the rods reaching into and being acted on by the compressive stresses in the beam, and, on the other hand, a loss due to the reduction of the length of the arm of the couple—the loss and gain are apparently about equal. Beams with part of the rods bent up and part left straight develop from 25 to 50% higher values than beams with the rods all straight. It is reasonable that this should be so. The concrete between the rods is increased from the beginning of the bend to the end of the beam, and such an increase has already been found to be a material factor in increasing the bond-friction-resistance. (See Fig. 6.)

Much depends on the manner of bending, but the most important point is to guard against sharp bends in the rods.

d.—Stirrups.—A series of tests on beams by the Boston Transit Commission,* is interesting because it shows that, as the proportion of concrete between the rods decreases, the value of the stirrups increases. Thus, of beams with $\frac{81}{100}$ of 1% reinforcement, those with stirrups were 25% better than those without stirrups; when 50% more reinforcement was put in similar beams, there was 50% increase in the value of the stirrups, that is, those with stirrups were 37% better

* Reported by Turneaure and Maurer.

than those without stirrups; and, when double the reinforcement was put in the beams, there was a twofold increase in the value of the stirrups, that is, those with stirrups were 56% better than those without stirrups.

In the Talbot series of 1907-08 the use of stirrups has been studied. The beams were 1 : 2 : 4 concrete, two months old, with cot. $\alpha = 2.8$. In plain-rod reinforcement, with $B = 1.38\ \Sigma O$, the average value for the beams with stirrups was 20% better than for those without stirrups. When $B = 1.03\ \Sigma O$, beams with stirrups were 18% better than those without. Beams with this value of B and having deformed-bar reinforcement were 26% better with than without stirrups. This illustrates in an entirely new manner the effect of the suspension of the concrete below the plane of the steel by the concrete between the rods. In this way, the stirrups are seen to perform the function which would otherwise be performed by a proportionate quantity of concrete.

e.—Combinations of Two or More of the Preceding.—One case, only, will be considered: the combination of stirrups with part of the rods bent up and part left straight. As previously mentioned, there does not seem to be any evidence of improvement over plain straight rods without stirrups. The only way the writer can account for the absence of improvement is in the possibility that the stirrups disturb the concrete around the region of the bent rod, thus destroying some of the advantage due to the influence of the compressive stresses on that bent rod. It is also probable that the absence of improvement is partly attributable to the fact that the reinforcement usually consisted of rods bent at sharp turns, instead of having continuous easy curves throughout.* Further investigation may throw new light on the question.

As to the practical aspect of the results of this investigation, with particular reference to the question of reinforced concrete design, a few points should be mentioned.

The problem of reinforced concrete design must be decided on purely scientific lines. In the writer's opinion, tests in reinforced concrete buildings are utterly impracticable and useless. Such tests are deceptive and misleading, because of many influences, more or less transitory, which aid the tested panel or series of tested panels so

* See "Some Mooted Questions in Reinforced Concrete Design," by Edward Godfrey, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXX, p. 54.

materially that, though incalculable and uncertain, they are nevertheless the dominating factors in carrying the loads. The loads themselves are seldom what they seem. It is necessary to understand the value of each element in a design, from a special study of that element. Its relation to the general problem must always be based on the judgment of the designer.

With regard to the question of loads in building work, it is the custom to specify a uniform distribution of loading. Buildings have been classed on the basis of the use to which they are to be put. The method of fixing the standard of classification has been by the determination of the uniform live load each class will safely carry. The precedents existing to-day as to what each class should be designed to sustain are based on a history of accidents. The lessons learned from these accidents have been so dearly bought that a designer must be bold indeed to presume to set up his judgment against the standard specified loadings for various classes of buildings. As previously mentioned, these standards are given in terms of the relative values for a uniform distribution of loading. Now, it can be shown that the comparative effect of a uniformly distributed load and a center concentrated load on a reinforced concrete beam is not the same as their comparative effect on a steel, a timber, or a plain concrete beam. It is commonly known that in the latter cases the effect of a concentrated load is double that of a uniform load on like beams; but in the case of a reinforced concrete beam, of the standard type, previously referred to, the bond-friction-resistance under a uniform load is not the same as under a concentrated load in the center of the beam. It is not unreasonable that this should be the case. For instance, consider the stresses produced on the cross-section midway between the center of the beam and the center of its supports. For uniform load, the strain on the cross-section is 75% of the strain at the center; and for center concentration of load, the strain is only 50% of the strain at the center (this is illustrated for any cross-section between the center and the support by Fig. 14), that is, uniform load will develop 50% more strain on the bond at the quarter-point than will be developed by center load. This is based on the assumption that the steel in each beam is strained alike at the center cross-section. Owing to the variable nature of the bond-friction-resistance, it would be an endless task to determine theoretically the location of the cross-section where

failure would occur on a reinforced concrete beam under a uniform distribution of load. However, experiments seem to indicate that failure will occur coincidently when like beams are loaded uniformly or are loaded at the center with 70% of the uniform load. This value, therefore, may be used as a basis for uniform loads when the values for the bond-friction-resistance given in Fig. 13 are under consideration.

The formulas for such a case would be developed as follows, when W = the total uniform load on the beam or girder:

$$M_b = \frac{70}{100}\frac{W\,l}{4} = 0.175\ W\,l$$

$$M_s = f_s\,A_s\,d' = f_b\,\Sigma O\,l'\,d'$$

$$\text{or } W\,l = 5.7\,f_b\,\Sigma O\,l'\,d'.$$

In this case l' will be half the length of the rod.

The size of the rod is an important factor in such reinforced concrete as depends on bond-friction-resistance to develop the tensile strength of the steel. This is plainly evident from the fact that, for equal areas of cross-section, the perimeter of rods or bars increases inversely as the diameter or side. This is true for round or for square sections. For instance, the perimeter of a 1-in. square bar is 4 in., the perimeter of a $\frac{1}{4}$-in. square bar is 1 in., and it requires sixteen bars per square inch of area, thus sixteen $\frac{1}{4}$-in. bars expose four times as much area of surface as a 1-in. bar. This is illustrated, for beams which have the real working dimensions most commonly used in buildings, in Figs. 15 and 16. In Fig. 15, it will be seen that, for the standard type of reinforcement, $\frac{1}{4}$-in. rods develop 64 000 lb. per sq. in. on an 18-ft. span uniformly loaded, when $B = \frac{2}{3}\,\Sigma O$; $\frac{1}{2}$-in. rods, 32 000 lb., and 1-in. rods only 16 000 lb. per sq. in. When $B = 2\ \Sigma O$ (Fig. 16), $\frac{5}{8}$-in. rods develop 67 000 lb.; 1-in., 42 000 lb., and 2-in. rods only 21 000 lb. per sq. in., on an 18-ft. span uniformly loaded.

With regard to the determination of the neutral axis and the centroid of compressive stress, the ordinary methods, as laid down by the Special Committee on Concrete and Reinforced Concrete, may be used for the working stresses on reinforced concrete. On the other hand, from the yield point of the bond, which occurs at about three-

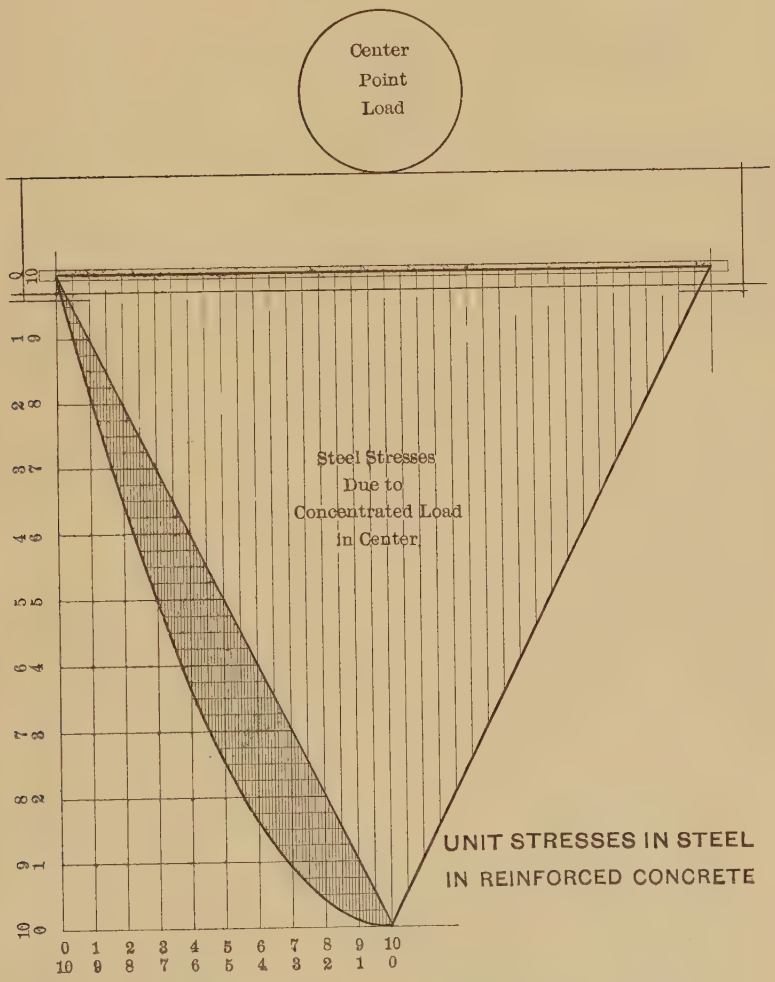

Center
Point
Load
Steel Stresses
Due to
Concentrated Load
in Center
UNIT STRESSES IN STEEL
IN REINFORCED CONCRETE
0 1 2 3 4 5 6 7 8 9 10
10 9 8 7 6 5 4 3 2 1 0
Increase in Steel Stresses
Due to an Equivalent Uniform Load

FIG. 14.

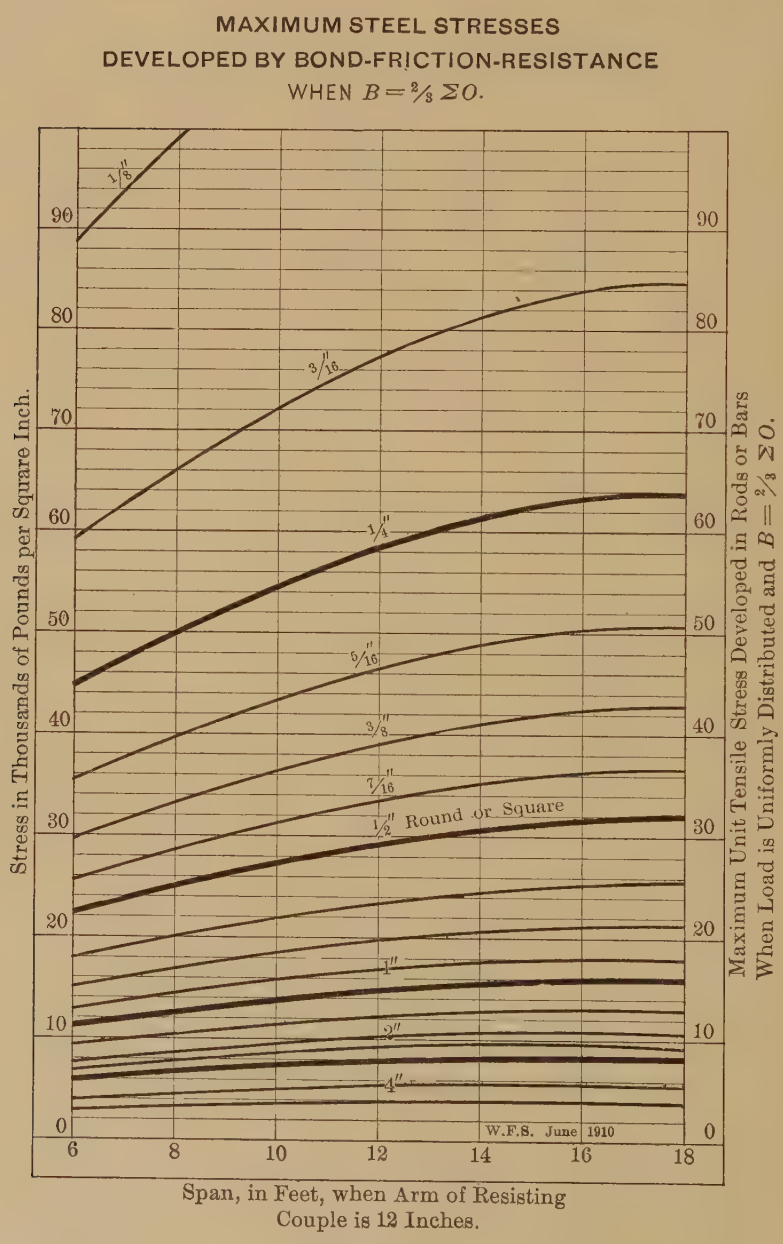

MAXIMUM STEEL STRESSES
DEVELOPED BY BOND-FRICTION-RESISTANCE
WHEN $B = \frac{2}{3} \Sigma O$.
Stress in Thousands of Pounds per Square Inch.
Maximum Unit Tensile Stress Developed in Rods or Bars When Load is Uniformly Distributed and $B = \frac{2}{3} \Sigma O$.
1/8"
3/16"
1/4"
5/16"
3/8"
7/16"
1/2" Round or Square
1"
2"
4"
W.F.S. June 1910
0
10
20
30
40
50
60
70
80
90
6
8
10
12
14
16
18
Span, in Feet, when Arm of Resisting Couple is 12 Inches.

FIG. 15.

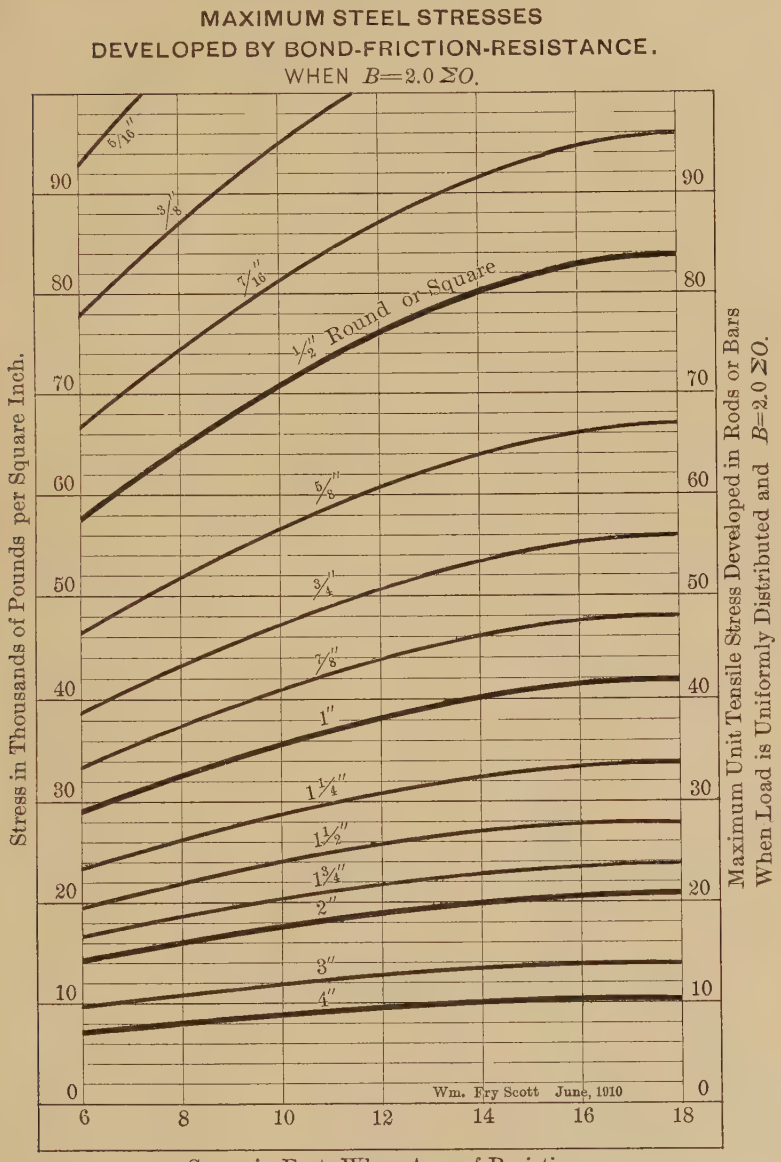

MAXIMUM STEEL STRESSES
DEVELOPED BY BOND-FRICTION-RESISTANCE.
WHEN $B=2.0\,\Sigma O$.
Stress in Thousands of Pounds per Square Inch.
Maximum Unit Tensile Stress Developed in Rods or Bars
When Load is Uniformly Distributed and $B=2.0\,\Sigma O$.
5/16"
3/8"
7/16"
1/2" Round or Square
5/8"
3/4"
7/8"
1"
1 1/4"
1 1/2"
1 3/4"
2"
3"
4"
0
10
20
30
40
50
60
70
80
90
6
8
10
12
14
16
18
Wm. Fry Scott June, 1910
Span, in Feet, When Arm of Resisting
Couple is 12 Inches.

FIG. 16.

eighths of the ultimate bond-friction-resistance, to the point of failure of the beam, the slipping of the rods has the effect of gradually raising the centroid of the compressive stresses, or, in other words, of increasing the length of the arm of the couple formed by the tensile and compressive forces. This increase at very near failure probably seldom exceeds 5 or 6% of the computed length of the arm.

In conclusion, the writer desires to say that at first glance the evidence presented herewith points to the fact that the stresses, in such reinforced concrete as depends on bond-friction-resistance to develop the tensile strength of the reinforcement, are much more complicated that has hitherto been supposed. This is true from a purely technical point of view only, and, notwithstanding the very valuable investigations of Messrs. Hatt, Talbot, Turneaure, Howe, Humphrey, Lanza, Marburg, Harding, and others, without whose work it would have been impossible to prepare this paper, and to whom the writer before closing wishes to express his indebtedness, there is a tremendous amount of work still to be done in engineering laboratories in the way of further investigation before the last word has been said on this subject. The Profession is especially indebted to those eminent investigators who have unselfishly reported full and comprehensive data on their experiments, thereby enabling any practicing engineer to draw his own conclusions from the material presented.

DISCUSSION

Mr. Godfrey.

EDWARD GODFREY, M. AM. SOC. C. E. (by letter).—Although this paper does not give specific conclusions for practical designing, it could be made a basis for such conclusions. As the writer is interested in reform in reinforced concrete construction, he would like to call attention to the fact that some of the author's conclusions, drawn from a critical study of tests, agree exactly with his own, which are based on a rational analysis of common practice, as set forth in his paper, "Some Mooted Questions in Reinforced Concrete Design."* As the writer is also interested in rational, that is, common-sense, analysis and correct logic, he would like to call attention to some errors, both original and borrowed, in Mr. Scott's paper.

In regard to the spacing of rods or bars, Mr. Scott quotes at length from Maurer and Turneaure,† and apparently with approval. It is surprising that he has not kept in closer touch with the literature on this subject. He refers later to the writer's paper in which the large error of this very quotation was expressly referred to, as well as the participation in the same by three other well-known authorities. The threefold error in this quotation has been pointed out by the writer.‡ It is simply this: If shear and bond stress are equal, the width of a beam must be equal to the sum of the perimeters of all horizontal reinforcing rods; for it is the width of the beam which measures the area in horizontal shear, and the sum of the perimeters of the rods which measures the area in bond stress. It is an error to state that if the clear space between rods be equal to that portion of their perimeters below a plane through their centers, shear and bond stress will be equal. This neglects the fact that the portion of the perimeter above the center of a rod is not equal to the diameter. This is the first error. It is also an error to say that the portion of the perimeter of a square rod with its sides vertical below a plane through the center, is equal to one and one-half diameters; it should be two diameters. This is the second error. Neither is the perimeter below the center of a square rod set diagonally one diameter; it should be two diameters. This is the third error.

The writer believes he is justified in going thus minutely into this matter, because it is an astounding fact that this and other errors, to some of which he has already called attention, work their way through the literature of the subject like "leaven that tends to leaven the whole lump." Writers should try to correct these invidious errors, rather than perpetuate them by approving references to them.

---

* *Transactions*, Am. Soc. C. E., Vol. LXX, p. 54.

† "Principles of Reinforced Concrete Construction."

‡ *Engineering News*, Nov. 11th, 1909.

Mr. Godfrey.

One important point made by Mr. Scott is his emphasis of the vital necessity of considering the relation of the width of a beam to the sum of the perimeters of the horizontal reinforcing rods.

He refers to the rule of the Special Committee on Concrete and Reinforced Concrete for spacing rods, as being in conformity with the theory heretofore referred to, which he quotes at such length. He fails to point out that this Committee, in the same paragraph of its report, allows a clear spacing between two layers of bars of not less than ½ in. This nullifies completely any semblance of "conformity with this theory." Mr. Scott missed an opportunity to characterize as monstrous, as it truly is, this authorized method of designing beams by crowding steel without limit into the bottom of a beam, when he found that his study of tests proved so conclusively the need of a large preponderance of concrete over steel and a wide spacing of rods.

In a large beam recently illustrated in an engineering periodical, there are shown five 1-in. round rods, five 1⅛-in. rods, and five 1¼-in. rods. The beam is 16 in. wide. The perimeters of these rods sum up to more than three times the width of the beam. None of the rods is anchored, except with a hook.

Mr. Scott says:

"Hooks on the ends of rods do not produce any increase in the strength of the reinforcement. They even tend to cause failure to occur sooner than when they are omitted."

He also says, "the most important point is to guard against sharp bends in the rods." The writer has long held these facts to be of the utmost importance. He emphasized them in his paper, before referred to, and holds them up in his book,* as elementary but neglected principles of correct design. His conclusion is based largely on rational and common-sense analysis. He is not, as one of the critics of his paper intimated, in the least concerned in being "placed right," but he is deeply concerned in seeing that the Engineering Profession is placed right in this and other vital matters of design. He would like, therefore, to add emphasis and endorsement to what Mr. Scott has said on these points.

The author's conclusions regarding stirrups appear to be about as follows: (1) They are not of much use except to help out a narrow beam; and (2) In a beam with some rods bent up, they are rather a hindrance than a help. These conclusions bear in the same direction as those of the writer, but they favor the stirrups unduly, because the conclusions are based largely on stirrups as a means of taking shear as against concrete alone, rather than against some rational means of shear reinforcement; and this is explained by the almost total absence in published tests of beams rationally reinforced for shear, that is, by curved-up rods anchored over the supports.

* "Structural Engineering: Concrete."

Mr. Scott deduces some of his theories, as to the internal stresses in a beam and their effect on the steel and concrete, from a totally erroneous premise. He gives a number of diagrams showing the actual and the apparent elongation of the steel in a reinforced concrete beam. The so-called actual elongation is in reality that calculated from the loading of the beam, and some of the complex theories of beams, which, strangely enough, assume a plane section after deflection, which Mr. Scott denies. The apparent elongation which is the actual stretch of the concrete beside the rod, as measured by extensometers, Mr. Scott finds to be much less under small loads and much greater under large loads than the "actual elongation." Mr. Godfrey.

This supposed difference in stretch of the rod and the surrounding concrete is explained by a theory which would appear to make of the concrete a material resembling gelatine in its properties. It is difficult to understand how the concrete could possibly stretch less than the steel, for it is the envelope of concrete which gives the stress, and, consequently, the stretch, to the steel. The concrete beam is loaded and deflected, and the steel relieves the concrete of part of its tension. It is also hard to see how a material like concrete could, in any event, stand the distortion incident to such differential stretch at its surface and at the surface of the steel, which may be but an inch or so beneath the surface of the concrete.

Of course, there will be a difference in stretch when the steel begins to slip, but the calculated stretch in the steel, estimated from its calculated stress and based on the complex and unsubstantiated formulas in common use, is a value quite incompetent to determine the point where the bond is broken.

Mr. Scott's formulas for finding the stress in the steel reinforcement do not take into account the tension in the concrete. Of course, a working formula for reinforced concrete beams must omit the assistance rendered by the tension in the concrete, but this does not annihilate that tension, and this is a potent argument against the validity of formulas which use such fine-grained theory and cast out vitally important considerations. A theory which finds the tension a plain concrete member would have to stand (either a beam, a chimney, or an arch), and then provides steel to take this tension, as advocated by the writer, is all that the case demands for practical designing.

Theories for the minute investigation of the internal working of a beam would have to include every factor, and the tensile strength of the concrete in a beam is a very large factor in the strength of a whole beam, though apt to be rendered *nil* by a single crack.

Neglect of the aid which the tensile strength of the concrete supplies in resisting transverse strains is a full explanation of the difference, which in some cases amounts to several hundred per cent., that

Mr. Godfrey.

Mr. Scott would find between the apparent elongation and the so-called actual (really calculated) elongation.

From his study Mr. Scott further concludes that the gripping power of the concrete increases as the distance of the load from the support decreases. This, too, is completely accounted for in the method of finding stress in the steel which he used. L. J. Johnson, M. Am. Soc. C. E., has described some tests,* and has given his conclusions therefrom. The loads were placed very near the supports, and the apparent gripping power of the concrete was very high. The writer† afterward pointed out that Professor Johnson's conclusions were wrong and misleading, because what he calculated as the high gripping power of the concrete could be fully explained by calculating the beam as plain concrete. On the basis of Professor Johnson's reasoning, that is, neglecting the tension in the concrete, the thinnest kind of wire could have been used, and an apparent grip of any desired amount could seemingly be found.

Suppose the ultimate resistance to bending, of a plain concrete beam, were just equal to the bending moment that causes slip in the steel of the same beam reinforced, when the load is in the center of the span. At the point where the concrete fails in tension the steel must take up all the tensile stress. Is it not clear, then, that if the steel is calculated to take all the tension, its apparent bond strength is in reality measured by the transverse strength of the plain concrete beam? As this latter is a constant, the apparent bond strength is exactly in the inverse ratio of the distance from the support to the loading point. Thus, if the steel rod slips when a certain load is central on the beam, it will of course slip when twice that load is placed in two portions at quarter points, because the bending moment on the concrete beam is just the same. This could be anticipated before tests are made. It is not in the least surprising.

It is just such errors as this, attributing to the steel reinforcement wonderful properties which, in fact, are contained in the concrete, which tend to place reinforced concrete engineering on a low level. Exploiters of patented systems, such as slab and beam reinforcement, claim properties which are contained in the concrete itself.

Mr. Scott finds that the tensile strength of the reinforcing steel shows up better when the load is near the middle of the span than when it is near the end. This follows from the same reasoning, for after the concrete is once cracked and the tension is all on the steel, the holding power will naturally be greater for the greater embedment, even though initial slip has taken place.

The author's conclusion as to the small, uncertain, and only occasional benefit derived from deformed bars agrees with what the writer

* *Journal*, Assoc. of Eng. Societies, June, 1907.

† *Engineering News*, August 29th, 1907.

has pointed out in his book on concrete. There is scarcely an occasion where deformed bars are of any real advantage. If the concrete is properly made and hardened, it will grip the steel with all the force necessary, assuming that the rods are not too large in section and that there is sufficient concrete surrounding the steel. These features are essential in any design. Mr. Godfrey.

The advantage apparent in test beams which set under water and which have deformed bar reinforcement, is explained by the fact that such concrete will not shrink and grip the steel. Of course, shoulders and deformities in rods will take some hold, but dependence on these, without grip, is precarious. Reinforced work which is to be under water should have the reinforcement riveted, or should have other end connections, entirely independent of grip.

The writer agrees with Mr. Scott as to the misleading character of tests on finished structures. He also believes that there is a large amount of work yet to be done. This work, however, which is sadly needed, is not so much the making of tests, as the correct interpretation of tests already made, and the breaking down of persistent and groundless errors in the way of false interpretations, false logic, and baseless deductions.

Joseph T. Maguire,* Esq. (by letter).—With reference to the first part of this paper, dealing with the slipping of rods, yield point of bond, and differences between "actual" and "apparent" elongations, it is an acknowledged fact, that "actual" elongation cannot possibly be calculated, unless the tension value of the concrete is definitely known, and even then the relative distribution of load between the concrete and steel would make it a difficult problem to solve. For this reason it would seem that the charts illustrated in Figs. 1 and 2 do not show the actual existing differences between the two elongations; consequently, their intersections would not give the true point of yield of bond. No attention whatever seems to have been given to any tension value of the concrete. The analysis of tests for the purpose of proving that the bond capacity of beams varies with the variations of cot. α seems to the writer to be fallacious, for the same reason, namely, that no attention has been paid to the tension value of the concrete. Mr. Maguire.

Take, for instance, the results on Beams Nos. 21 and 5 of the Talbot series of 1905. A calculation of the former beam shows that it developed a bond strength of 88 lb. per sq. in. The actual failure of the beam was by tension in the steel, therefore it is safe to assume that its actual bond capacity was in excess of 88 lb. per sq. in. This beam was loaded at the center, and cot. $\alpha = 8.4$.

Beam No. 5, loaded at the one-third points, with cot. $\alpha = 5.6$, failed by the tension in the steel, and developed a bond strength of

* Bureau of Yards and Docks, Navy Dept., Washington, D. C.

Mr. Maguire.

103 lb. per sq. in. This bond strength, of course, was confined to that portion of the rod with varying bending moment, that is, from the point of support to the point of application of the load. If, instead of assuming that the variation in the position of the load created a variation in the capacity of the bond strength, it is assumed that the vertical concrete plane immediately under the load has value in tension, it can be demonstrated, quite readily, that, with an assumed value of 90 lb. bond strength in the rod, there is approximately 100 lb. tension in the extreme fibers of the concrete plane. As the value of concrete in tension is known to be considerably in excess of this, one can readily see why the beam did not fail by diagonal tension, and why the apparent bond was so high. Of course, had incipient cracks developed in the concrete at this plane of tension, due to the elongation of the steel, thus destroying its tension value, this line of reasoning would be fallacious, but no evidence has been adduced to prove the presence of these cracks at the crucial points.

Bond is a function of the molecular construction of the beam, and the writer fails to see how a modification of extraneous conditions can possibly affect its capacity. The method of analysis suggested above will explain satisfactorily every one of the tests used as a basis for the creation of the diagrams with varying cot. α, without admitting for a moment that the variation in the application of the load varies the capacity for bond.

Take, for instance, Beam No. 20 of the Talbot series of 1905: This beam was loaded at the one-third points, and failed by diagonal tension. It has the same cot. α as Beam No. 5, yet it developed only a bond of 93.4 lb. per sq. in., and failed by diagonal tension, whereas Beam No. 5 developed a bond strength of 103 lb., and then failed by tension in the steel. On the basis of the argument suggested by the author, these bond values should have been nearly equal, and no explanation is offered for the discrepancy. On the basis of the analysis presented herewith, the reason for the failure of this beam, with such a low bond strength, was the presence of an incipient crack, just at or near the point of loading, which destroyed the tension value of the concrete. This seems to be a much more logical explanation than that offered by the diagrams.

Beams Nos. 11, 13, 16, 17, etc., grouped in the original diagram, follow the same analysis as that suggested for Beam No. 5.

Beams Nos. 14, 15, and 18 failed by diagonal tension, and gave tension, on the outer face of the concrete plane, of about 280 lb. per sq. in., with an assumed bond capacity of 90 lb. per sq. in., which would account for their failure in diagonal tension.

With respect to the values of bond-friction-resistance, with variable ratio of summation of perimeters of rods to breadth of beam, the

writer respectfully submits that no such table is capable of being created, unless the tension values of the concrete are known. Mr. Maguire.

In the foregoing discussion where mention is made of tension in the concrete plane, and failure of the beam due to the same, at or near the point of loading, this is understood to be simply theoretical, as the beams may have failed by tension in the concrete to the left of the point of application of the load. The point intended to be made is simply this: When the increment of bond which the rods fail to compensate reaches a point beyond the value of the concrete in tension, the beam fails. This point may be anywhere on the beam, and will be determined by the position of the incipient cracks.

The writer submits the following calculations on beams of Professor Talbot's tests, showing bond, etc.:

Beam No. 21, 1905.—Loaded at center. Failed by tension in steel. Beam 8 in. wide, 10 in. deep to steel, and with neutral axis at 0.420 d from top of beam. Span = 12 ft. 0 in. Reinforcement, four $\frac{1}{2}$-in. rods. Center load = 9 500 lb. Bending moment, neglecting weight of beam, = 342 000 in-lb. Tension in steel, due to same, = about 51 000 lb. (approximate check on Professor Talbot's tests). Bond stress on rod = 88 lb. (still neglecting weight of beam), which checks Mr. Scott. This beam gave by tension in the steel, therefore it is safe to say that the bond was something in excess of 88 lb. per sq. in. Cot. $\alpha = 8.4$.

Beam No. 20, 1905.—Failed by diagonal tension. Same dimensions, reinforcement, and span as Beam No. 21. Loading at one-third points, 5 000 lb. at each point. Bending moment (without weight of beam) = 240 000 in-lb. Neutral axis, 0.445 d from top; $d = 8.52$ in. Tension in steel = 35 900 lb., which checks Professor Talbot, approximately. End bond, disregarding weight of beam, = 93.4 lb. per sq. in. This value may or may not be high for the bond, depending on whether or not tension cracks had been developed at the point of rupture of the beam by the elongation of the steel. Only a knowledge of everything that happened during the test could determine this. Cot. $\alpha = 5.6$.

Beam No. 5.—Failed by tension. Same dimensions. Load, 11 000 lb. at one-third points. Neutral axis, 450 d; $d = 8.5$ in. Moment (disregarding weight of beam) = 264 000 in-lb. Tension in steel = approximately 40 000 lb. per sq. in. Bond = 103 lb. per sq. in. over the one-third length of rods from point of support to point of loading.

Now, assuming that the bond was good for 90 lb. per sq. in., there is an unbalanced bond of 3 921 lb. This would give a tension on the outer fibers of the plane of the concrete (at the point where the bond changes from apparently 103 lb. to 0 lb.) of about 103 lb. per sq. in., which is away within safe limits, unless, as stated for Beam No. 20,

Mr. Maguire.

incipient cracks had developed at this point, thus destroying the tension value of the concrete plane.

It will be noticed that Beams Nos. 5 and 20 were of the same dimensions, with absolutely the same reinforcement, and yet, Beam No. 20, with an applied load of 10 000 lb. at the one-third points, failed by diagonal tension (bond failure, really), whereas Beam No. 5 failed by tension with an applied load of 11 000 lb. at the one-third points. The bond in one case was 93.5 lb., in the other it was 103 lb., while cot. α remained constant. This can be readily explained by the method of analysis presented herewith.

Beams Nos. 11, 13, 16, 17, 19, 23, and 26.—These are grouped with Beams Nos. 5 and 20, on Fig. 4, and are similar. All failed by tension; therefore they follow the general analysis indicated for Beam No. 5.

Beams Nos. 31 and 50.—These are in the same group, and were dissimilar only in the application of loads; they follow the same analysis.

Beam No. 30.—Same dimensions. Loaded at center, with 7 000 lb. Failed by tension in the steel. Bending moment = 252 000 in-lb. (without weight of beam). $d = 8.566$ in. Tension = approximately 37 500 lb. Bond = 65 lb. per sq. in. Of course, the real value was in excess of this, as the beam failed by tension in the steel.

Beam No. 14.—Load, 15 000 lb. at points $7\frac{1}{2}$ ft. apart. Bending moment = $7\,500 \times 27 = 202\,000$ in-lb.; $d = 7.99$ in. Bond = 150 lb. per sq. in. Tension in concrete, allowing bond of 90 lb. per sq. in., = 282 lb. on the outer fiber. This beam failed by diagonal tension, due, in all likelihood, to failure of the tension plane referred to in the previous discussion.

Beams Nos. 15 and 18.—These gave results similar to those of Beam No. 14, though slightly higher.

Mr. Talbot.

A. N. Talbot, M. Am. Soc. C. E. (by letter).—In reading this paper, there is a feeling of disappointment in finding that it is based on such fallacious reasoning. The main thesis of the paper—the so-called new laws—has no logical connection with the data of the beam tests cited. The values given in the elaborate diagram, Plate XV, and in Fig. 13, are based on the calculated bond stress developed in a number of beam tests, but the author does not attempt to show that the bond stress was the cause of the failure, or even that it was connected with the failure, or influenced the loads carried by the beam. Naturally, for beams failing by tension in the reinforcing bars in the middle portion of the beam (as was the case with most of the beams referred to in Fig. 4), the loads carried will be greater as the load points approach the ends of the beam, and, therefore, the bond stress developed will be greater, but the author does not present any evidence that the bars slipped or that the condition of the bond stress with reference to a yield point of bond resistance affected in any way the load carried by the beam. Without such evidence the usual

Mr. Talbot.

methods of analysis would be followed, and these indicate that the amount of bond developed in the beam tests cited have no more relation to the loads which the beam carried than does the percentage of reinforcement bear to the amount of compression strength which may be developed in a beam. As the whole structure of the argument is based on a fallacy, the further deductions are also without foundation, such as those on the influence of the angle, α, of the diagrams, and the relation of periphery of bars to width of beam.

Under the circumstances it does not seem necessary to discuss other parts of the paper further than to give examples of the many faulty statements presented. The lines given as "actual elongation" in Figs. 1 and 2 are said to have been "computed from the known value of the modulus of elasticity of the steel and from the value of the tension produced therein as calculated from the known forces acting on the specimen," but the elongation of the steel itself was not measured, and there is every reason to believe that this elongation, at least through the middle portion of the beam, does not differ greatly from the actual elongation of the concrete alongside of it, and, therefore, that the author's line is greatly in error. The statement on page 232 (made without evidence to support it), that the first phenomena noted were a creeping of the rods, etc., is misleading, and the assertions made on page 234 about the apparent slip being equal to the actual elongation of the rod, are absurd. Whatever relation there may be between bond resistance and normal pressure, the fanciful conception of a normal component to the bond stress given in Fig. 3 is not borne out by any tests which the writer has seen. The localized high elongations in steel at stresses well beyond the yield point have no bearing on the distribution of stresses in a beam, up to the yield point of the steel, either at the middle or outside the load points where bond stress is more highly developed. The greater bending moment developed with a single concentrated load than with distributed loads is not explained by the author's statements; it is easily explained on other grounds. The author avoids giving the value of the angle, α, which should be taken for a beam with a uniform load. It would also be interesting to know the author's basis for the statement that the yield point for bond resistance is three-eights of the ultimate resistance regardless of the condition of the surface of the bar.

Any definite information throwing light on the action of reinforced concrete should be welcomed. Many matters are but imperfectly known. The action of the bond resistance between the steel and the concrete is a good subject for tests. The distance between bars and the thickness of concrete around the bars necessary to develop sufficient bond resistance under various conditions of loading and of bending up of bars, is an interesting topic for investigation and discussion. Under some conditions there is an intimate relation between

Mr. Talbot. the method of failure and the bond stresses, shearing stresses, and other stresses developed, and in such cases the conditions are very complex. It is hoped that fuller information on such matters will be obtained as time goes on. It is all the more unfortunate, then, that a paper should be presented having fallacies which invalidate the whole discussion. Fortunately, these are so patent that the careful reader should not be misled.

Mr. Burroughs. HECTOR ROBINS BURROUGHS, ASSOC. M. AM. SOC. C. E. (by letter).—This paper was read by the writer with special interest, inasmuch as he has been intending to present a paper of this character, which would throw some light on a subject which seems to be much in the dark, so far as he has learned from experience and association with men designing work of this class.

In reading the paper, the writer was impressed with the low values given, and on first thought it seemed that the bond-friction-resistance in a reinforced concrete beam is not consistent with the other allied elements of resistance. Under certain conditions and assumptions, this is believed to be true. Recently he had occasion to design several reinforced concrete slabs, which were supported on the tops of steel I-beam stringers in city highway bridges. After some study and discussion on the subject of bond stress, the writer adopted the formula:

$$v = \frac{V}{b' jd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

for calculating the horizontal shear, from which the bond stress

$$= \frac{V}{jd\, \Sigma O} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in a beam, was derived, and assumed at 80 lb. per sq. in., as a maximum working value.

In Formulas 1 and 2:

V = Total vertical shear at any section,
b' = Width of web of T-beam,
jd = Arm of resisting couple for any beam,
O = Perimeter of rod.

The specified maximum live load affecting the slab, was given as a 12-ton road roller. The stringers were generally 5 ft. from center to center, which was governed by the fact that it was likely that future trolley tracks would be required, and they were made of sufficient strength for that work.

In order to satisfy Formula 2 by the assumed allowable stress, it was found necessary to make the slabs considerably deeper and use more steel than the flexural stresses required. In other words, the percentage of steel depended on bond-friction-resistance, and was increased to 25 or 50% more than that required by flexure. It was possible to make the two required areas practically the same, by using

Mr. Burroughs.

smaller rods with closer spacing; but, as the rods were placed in the top and also in the bottom of the slab, the spacing was thought to be too close to permit of satisfactory tamping.

The writer understands that the author gives average values along the length of the bar. In a beam uniformly loaded, the bond-friction-resistance of the bar at the center would be equal to the sum of the perimeters multiplied by one-half the length of the bar, multiplied by a value taken from Fig. 13. In Formula 2 the bond-friction-resistance depends on the shear or its function, bending increment, which is a parabola for a uniformly distributed load, and hence the movement of bond-friction-resistance increases accordingly toward the support. The author shows that the bond-friction-resistance increases as it approaches the support, and is therefore in direct relation to the stress. The values given in the paper being average values for the embedded length of the rod, it would seem that the actual values near the support were considerably greater than those given, and are the values desired for computation when applying Formula 2.

The problem of computing bond-friction-resistance seems to be analogous to that of rivet spacing in a plate girder.

Comparing the results obtained by using allowable values from Fig. 13 multiplied by the embedded length of rod, multiplied by the sum of the perimeters, with the results obtained from Formula 2, will show from 25 to 50% greater resistance in favor of the results from the values in Fig. 13. This amounts to increasing the assumed maximum allowable value of 80 lb. per sq. in. The writer believes that the increment of resistance is the critical condition. If 80 lb. per sq. in. is a low value, what is a maximum allowable value?

It seems clear that incipient cracks are produced by increments of stress, and considering that the full strength of the beam is only partly realized, still its integrity is appreciably impaired, when these cracks (which are likely to admit moisture and cause corrosion) appear.

The writer regrets that maximum allowable values, in addition to average values, were not obtained, in order that the generally accepted Formula 2 could be directly answered, by assigning safe values for bond-friction-resistance, where the shear is a maximum.

In continuous and simple beams the maximum shear is at the support. In continuous beams the bending moment is negative, and hence the bond-friction-resistance for tension is along the continuity rods, and is a maximum at the support. It may be seen that in short spans, where the bond-friction-resistance is the determining factor for steel percentage, the greatest amount of steel will obtain over the support, and the maximum positive bond-friction-resistance in the lower rods will be just beyond a point of contraflexure toward the center of the span, for a beam over two or more supports.

Mr. Burroughs.

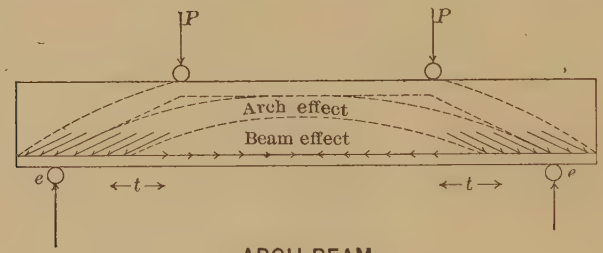
The author states that hooks at the ends of the rods did not increase the bond-friction-resistance. It seems clear that this might be expected, since bond-friction-resistance is measured by increments. The ultimate strength of a beam, after the bond had failed, would probably be increased by hooks and other similar anchors, the action changing from beam action to that of an arch.

Considering that deformed bars did not show an appreciable advantage over plain bars, the writer agrees with the author that they probably would with equal care in tamping. It may be that the so-called "chemical action," between concrete and the ordinary round rod is the cause of this anomaly and is not yet fully understood.

Mr. Frye.

ALBERT I. FRYE, M. AM. SOC. C. E. (by letter).—This paper is a most interesting and instructive contribution to the practical study of the action of reinforced concrete beams.

Of electricity, it has been said that "we don't know what it is, but we know what it will do;" but, with a reinforced concrete beam, we do not even know what it will do—under varying conditions. If any



ARCH-BEAM

FIG. 17.

doubt exists on this point it will certainly be dispelled by a careful perusal of the recent report of the Special Committee appointed by the Society to take up the question of concrete and reinforced concrete. After considering the matter for about five years, Majority and Minority Reports were presented.* The writer believes that the Majority Report is excellent, as far as it goes, but the working beam formulas are defective in recognizing a reinforced concrete beam as a beam, pure and simple.

Those who have attempted an analysis of the various column formulas—and perhaps have framed some of them—have not failed to realize that the practical column is a sort of hybrid: a combination of the ideal long column which tends to bend, and the short strut which tends to shear and buckle. Similarly, the reinforced concrete beam, and especially the short beam of the laboratory type, is really a combination of beam and arch, that is, an arch-beam. Fig. 17 illustrates such a beam, showing the arch effect and the beam effect. Any

* *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 431.

formula, even if it could be devised, which would attempt to represent accurately the combined arch-beam effect on bond, shear, and bending, would be too long and intricate to be of any practical use; but the writer will attempt to show that the so-called arch-beam formulas can be represented, for all practical purposes, by the pure beam formulas, with certain constants in the latter changed to variables, to meet the variable conditions of loading. Mr. Frye.

Experiments show, as the author points out, that the bond-friction-resistance increases as the loads, *P*, Fig. 17, approach the points of support of the "beam," and the writer believes that this is partly due to the arch effect, partly relieving the beam of its purely beam function. As an arch, it is partly supported along the central portion (where the lower concrete fibers are weakened by tension, thus weakening the bond), and partly restrained at the ends by the rods and the bond forces at those points. Furthermore, in the pure beam theory, it is assumed that no portion of the concrete fiber is in tension; whereas, it is known that tension actually exists in the lower fibers throughout some portion near either point of support, as at *t t*, Fig. 17. Also, the slipping of the rods in the concrete, toward the center of the beam, tends to produce compression in the lower concrete fibers along the overhang, *e*, and for some distance on the span proper, next to the supports.

In view of the foregoing discussion, and the results of many tests, the writer believes that the allowable bond-friction-resistance used in beam formulas should be a variable quantity, and that some formulas like the following will approximately represent the safe allowable bond-friction-resistance, *B*, for any kind of loading on the beam:

$$B = B_1\left(1 - \frac{1}{2} \cdot \frac{\text{average } u}{\text{maximum } u}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

In which B = allowable bond stress, in pounds per square inch;

B_1 = bond stress for least dangerous loading;

$$u = \frac{V}{jd\ \Sigma O} = \text{bond stress per square inch of bar} \quad \ldots\ldots (2)$$

Average u = average throughout the beam;

Maximum u = maximum at left support;

V = total shear, in pounds;

jd = arm of couple, in inches;

ΣO = sum of perimeter of bars, in inches.

If the bar is of uniform section throughout, the shear, *V*, can be substituted for *u*, and Formula 1 reduces to

$$B = B_1\left(1 - \frac{1}{2} \cdot \frac{\text{average } V}{\text{maximum } V}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Assuming $B_1 = 100$, the following is the working formula:

$$B = 100\left(1 - \frac{1}{2} \cdot \frac{\text{average } V}{\text{maximum } V}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Mr. Frye.

Figs. 18, 19, and 20, with the tables at their left, give the respective values of B for a single concentrated load, twin loads, and uniform loading, based on Formula 4. It will be noted that the values of B range from 50 to 100, with average values of 75 lb. per sq. in.; and that the calculated values of u, from Formula 2, should not exceed these values of B for the loadings indicated.

It is also suggested, that, if the pure beam formulas are used, the values of f_s and f_c, due to moments, do not represent correctly the fiber stresses in the steel and concrete; but that some formulas like the following, which give variable values, due to different kinds of loading, would be more approximately correct:

For beams of uniform cross-section,

$$F_s = 10\,000\left(1 + \frac{\text{average } M}{\text{maximum } M}\right), \text{ and } F_c = 400\left(1 + \frac{\text{average } M}{\text{maximum } M}\right) \quad (5)$$

In which F_s = allowable tensile fiber stress for steel;
F_c = allowable compressive fiber stress for concrete;
M = bending moment.

In the following formulas,

$$f_s = \frac{M}{p\,jd\,bd}, \text{ and } f_c = \frac{2\,M}{jd\,kd\,b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

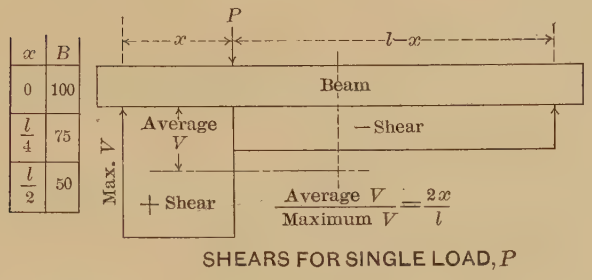
f_s must not exceed F_s, nor f_c exceed F_c, obtained from Formulas 5.

TABLE 2.—Allowable Fiber Stresses, F_s and F_c, in Beams for Various Kinds of Loadings.
(See Formulas 5 and 6.)

Values of $\frac{x}{l}$	For concentrated load.		For twin loading.		For uniform load.	
	F_s	F_c	F_s	F_c	F_s	F_c
0	15 000	600	20 000	800	16 667	667
$\frac{1}{8}$	15 000	600	18 750	750	16 667	667
$\frac{1}{4}$	15 000	600	17 500	700	16 667	667
$\frac{3}{8}$	15 000	600	16 250	650	16 667	667
$\frac{1}{2}$	15 000	600	15 000	600	16 667	667

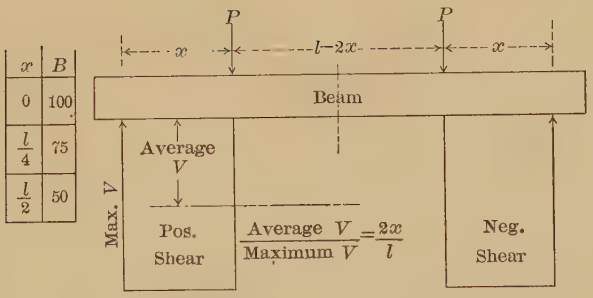
Table 2 gives the respective values of F_s and F_c for a single concentrated load, twin loads, and uniform loading on a beam, based on Formulas 5. It will be noted that the values of F_s range from 15 000 to 20 000, with average values of about 17 000 lb. per sq. in.;

Mr. Frye.



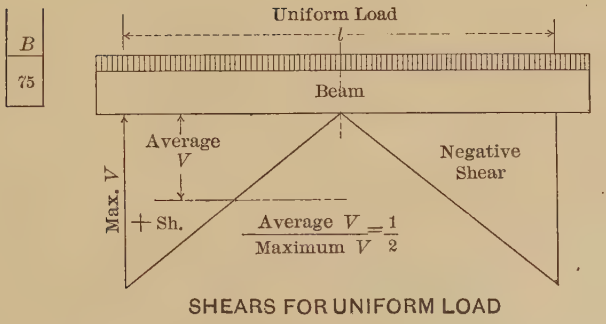
SHEARS FOR SINGLE LOAD, P

FIG. 18.



SHEARS FOR TWIN LOADS, P

FIG. 19.



SHEARS FOR UNIFORM LOAD

FIG. 20.

Mr. Frye. and the values of F_c range from 600 to 800, with average values of about 675 lb. per sq. in.

Among the most instructive experiments on bond-friction-resistance, in addition to those cited by the author, are the tests conducted at the laboratory of the Technical College at Darmstadt, and reported in *Armierter Beton* for September, 1910.

The Darmstadt tests were on 12 by 12-in. beams, 82 in. long, with a clear span of 74 in., and were subjected to incremental twin loadings at the quarter-points, that is, 18½ in. from either support, Fig. 21. A single rod, 0.94 in. in diameter was embedded ¾ in. from the bottom of each beam. Three beams were tested. The slipping of the rods was measured at four points, namely: at the center of the beam, point (5); at the left-hand quarter-point, (4); at the left support, point (2); and at the end of the beam, point (1), 4 in. from the support.

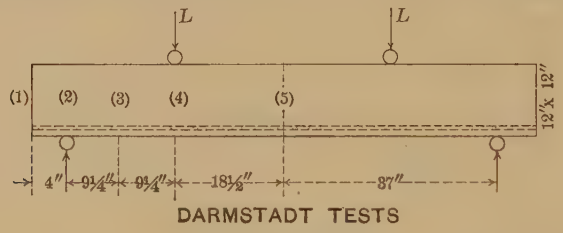


FIG. 21.

An analysis of these tests indicates that the bond-friction-resistance was considerably weakened or reduced at points where the tensile stresses in the concrete had become high, and perhaps broken the concrete fibers and produced the usual cracks. At even moderate loads the slipping of the rods in the concrete at the quarter-points became quite marked, and indicated a readjustment of stresses not in strict conformity with the beam theory. As the loads were increased the bond-friction-resistance was quite wholly destroyed for the major middle section of the beams, the beams being resolved into arch-beams, as previously explained.

Table 3 gives a summary of the results of the Darmstadt tests, showing the slip of the rods (in hundredths of an inch) for various incremental loads. After the twin loads, L, were applied they were reduced to one-tenth of a metric ton before each successive application. The three beams are designated as A, B, and C. The foot-notes to the table record the appearance of cracks. The numbers in the first column indicate the loadings, and correspond with the numbers of similar loadings in Tables 4 and 5.

Table 4 gives the calculated values of the Darmstadt beams, for v (the shearing stress, in pounds per square inch) and u (the bond stress, in pounds per square inch) from the formulas presented in the Majority Report of the Special Committee on Concrete and Reinforced Concrete.

TABLE 3.—Darmstadt Beam Tests.

Slip measurements, in hundredths of an inch.

Note: (+), Rods slipped to the right.
(—), Rods slipped to the left.
(See Tables 4 and 5.)

No.	Twin loads, *L*, each.		Slip measurements, in hundredths of an inch, at—											
			End of beam, Point (1).			Left support, Point (2).			Point of loading, Point (4).			Center of beam, Point (5).		
			Three tests, beams:			Three tests, beams:			Three tests, beams:			Three tests, beams:		
	In metric tons.	In pounds.	*A*	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*
	0.1	220	.....	.....	.....	0	0	0	0	0	0	0	0	0
1	0.5	1 102	.....	.....	.....	0	0	0	0	0	0	0	0	0
	0.1	220	.....	.....	.....	0	0	0	0	0	0	0	0	0
2	1.0	2 204	.....	.....	.....	0	0	0	+0.008	0	0	+0.008	−0.004	+0.004
	0.1	220	.....	.....	.....	0	0	0	+0.004	+0.004	0	+0.004	0	+0.004
3	1.25	2 755	.....	.....	.....	0	−0.004	0	+0.012	0	−0.008	+0.012	−0.008	+0.008
	0.1	220	.....	.....	.....	0	0	0	+0.012	+0.004	−0.008	+0.012	0	+0.004
4	1.5	3 306	.....	.....	.....	0	−0.004	0	+0.012	+0.004	−0.012	+0.020	−0.008	+0.016
	0.1	220	.....	.....	.....	.....	0	0	+0.008	+0.004	−0.012	+0.016	−0.004	+0.008
5	1.75	3 857	.....	.....	.....	+0.004	−0.004	0	−0.004	0	−0.020	+0.031	−0.016	+0.020
	0.1	220	.....	.....	.....	0	−0.004	0	0	0	−0.012	+0.020	−0.012	+0.012
6	2.0	4 408	.....	.....	0	+0.004	0	0	−0.020	−0.016	−0.028	+0.043	−0.031	+0.028
7														
	0.1	220	.....	.....	.....	0	0	0	−0.004	−0.004	−0.016	+0.020	−0.020	+0.016
8	2.25	4 959	.....	+0.020	.....	+0.004	+0.012	.....	−0.028	−0.067	.....	+0.051	−0.055	.....
9														
	0.1	220	.....	.....	.....	0	+0.020	.....	+0.004	−0.017	.....	+0.024	−0.028	.....
10	2.50	5 510	0	+0.512	.....	+0.008	+0.524	+0.012	−0.047	−0.205	−0.028	+0.063	−0.118	+0.055
	0.1	220	.....	.....	.....	+0.004	+0.571	0	+0.004	−0.012	−0.020	+0.028	−0.055	+0.008
11	2.75	6 061	.....	.....	0	+0.016	.....	+0.004	−0.071	.....	−0.043	+0.075	.....	+0.083
12														

TABLE 3.—(*Continued.*)

No.	Twin loads, *L*, each.		Slip measurements, in hundredths of an inch, at—											
			End of beam, Point (1).			Left support, Point (2).			Point of loading, Point (4).			Center of beam, Point (5).		
	In metric tons.	In pounds.	Three tests, beams:			Three tests, beams:			Three tests, beams:			Three tests, beams:		
			A	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*
	0.1	220	.....	.....	.....	+0.016	.....	0	+0.008	.....	−0.031	+0.028	.....	+0.043
13	2.875	6 336	.....	+2.99	.....	.....	+3.37	.....	.....	−0.276	.....	.....	−0.150	.....
	0.1	220	.....	+3.11	.....	.....	+3.44	.....	.....	−0.004	.....	.....	−0.063	.....
14	3.0	6 612	+0.472	.....	+0.020	+0.582	.....	+0.004	−0.106	.....	−0.118	+0.083	.....	+0.122
15														
	0.1	220	+0.512	.....	.....	+0.642	.....	+0.012	+0.016	.....	−0.039	+0.031	.....	+0.071
16	3.5	7 714	.....	.....	+1.26	.....	.....	+0.480	.....	.....	−0.186	.....	.....	+0.276
	0.1	220	.....	.....	.....	.....	.....	+0.524	.....	.....	+0.047	.....	.....	+0.083
17	3.7	8 155	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Notes on Table 3.

		Beam.	Cracks appeared:	Remarks.
10	2.5	*A*	At right load point (4)	.....
10	2.5	*B*	At left load point (4)	.....
11	2.75	*A*	2 cracks: at and near left load point (4)	.....
11	2.75	*C*	3 cracks: at center of beam, point (5), and near each load point (4)	.....
13	2.875	*B*	.....	Loads fell to 2.75 tons.
14	3.0	*A*	Just to right of right load point (4)	Loads fell to 2.8 tons.
16	3.5	*C*	Increase of cracks under left load point (4)	Loads fell.

TABLE 4.—Darmstadt Beam Tests.

Calculated values of v and u, in pounds per square inch.

(See Table 3.)

No.	Twin loads, L, each.		Calculated values of v and u at—						Average values for total length of beam, 82 in.		Average values for total length of span, 74 in.	
			Left support, Point (2).		One-eighth point, Point (3).		Point of loading (quarter-point), Point (4).					
	In metric tons.	In pounds.	v.	u.	v.	u.	v.	u.	v.	u.	v.	u.
1..	0.5	1 102	13.6	55.3	12.6	51.2	11.6	47.1	6.1	25.0	6.8	27.7
2..	1.00	2 204	23.1	94.2	22.1	90.1	21.1	86.0	10.4	42.5	11.6	47.1
3..	1.25	2 755	27.9	113.7	26.9	109.6	25.9	105.5	12.6	51.3	14.0	56.9
4..	1.5	3 306	32.7	133.1	31.7	129.0	30.7	124.9	14.7	60.1	16.3	66.6
5..	1.75	3 857	37.5	152.5	36.5	148.4	35.5	144.3	16.9	68.8	18.7	76.3
6..	2.00	4 408	42.3	172.0	41.3	167.9	40.3	163.8	19.1	77.6	21.1	85.9
7..	2.245	4 948	46.9	191.1	45.9	187.0	44.9	182.9	21.2	86.2	23.5	95.6
8..	2.25	4 959	47.0	191.5	46.0	187.4	45.0	183.3	21.2	86.4	23.5	95.7
9..	2.409	5 309	50.1	203.9	49.1	199.8	48.1	195.7	22.5	92.0	25.0	102.0
10..	2.5	5 510	51.8	211.0	50.8	206.9	49.8	202.8	23.4	95.2	25.9	105.5
11..	2.75	6 061	56.6	230.4	55.6	226.3	54.6	222.2	25.5	103.9	28.3	115.2
12..	2.796	6 162	57.5	234.0	56.5	229.9	55.5	225.8	25.9	105.6	28.8	117.0
13..	2.875	6 336	59.0	240.1	58.0	236.0	57.0	231.9	26.6	108.3	29.5	120.0
14..	3.000	6 612	61.4	249.9	60.4	245.8	59.4	241.7	27.6	112.7	30.7	124.9
15..	3.046	6 713	62.3	253.4	61.3	249.3	60.3	245.2	27.7	114.3	31.1	126.7
16..	3.5	7 714	71.0	288.8	70.0	284.7	69.0	280.6	32.0	130.2	35.5	144.4
17..	3.7	8 155	74.8	304.3	73.8	300.2	72.8	296.1	33.7	137.3	37.4	152.1

TABLE 5.—Darmstadt Beam Tests.

Calculated values of f_s and f_c, in pounds per square inch.

(See Table 3.)

No.	Twin loads, L, each.		Calculated values of f_s and f_c, at—						Average values for total length of beam, 82 in.		Average values for total length of span, 74 in.	
			One-eighth point, Point (3).		Point of loading (quarter-point), Point (4).		Center of beam, Point (5).					
	In metric tons.	In pounds.	f_s.	f_c.	f_s.	f_c.	f_s.	f_c.	f_s.	f_c.	f_s.	f_c.
1	0.5	1 102	2 075	68	4 006	131	4 326	141	2 832	92	3 139	102
2	1.	2 204	3 604	118	7 064	231	7 384	241	4 901	160	5 432	177
3	1.25	2 755	4 368	143	8 593	281	8 913	291	5 936	194	6 578	215
4	1.5	3 306	5 133	168	10 122	331	10 442	341	6 971	227	7 726	252
5	1.75	3 857	5 898	193	11 651	381	11 971	391	8 005	262	8 872	290
6	2.	4 408	6 662	218	13 180	431	13 500	441	9 040	295	10 019	327
7	2.245	4 948	7 412	242	14 679	480	15 000	490	10 054	329	11 143	364
8	2.25	4 959	7 426	243	14 709	481	15 029	491	10 075	330	11 165	365
9	2.409	5 309	7 912	258	15 679	512	16 000	523	10 733	350	11 895	388
10	2.5	5 510	8 191	268	16 238	531	16 558	541	11 110	362	12 313	402
11	2.75	6 061	8 956	293	17 767	581	18 087	591	12 145	397	13 459	440
12	2.796	6 162	9 096	297	18 047	590	18 367	600	12 335	403	13 670	446
13	2.875	6 336	9 338	306	18 531	606	18 851	616	12 662	413	14 033	458
14	3.	6 612	9 720	318	19 296	631	19 616	641	13 179	430	14 606	477
15	3.046	6 713	9 861	322	19 577	640	19 898	650	13 369	437	14 817	484
16	3.5	7 714	11 249	368	22 354	731	22 674	741	15 249	498	16 899	552
17	3.7	8 155	11 851	388	23 576	771	23 896	781	16 077	525	17 817	582

Mr. Frye. Similarly, Table 5 shows the calculated values for f_s (the tensile stress in the steel, in pounds per square inch) and f_c (the compressive stress in the concrete, in pounds per square inch), from the same formulas. Average values for v, u, f_s, and f_c are also given. The weight of the beams is assumed at 150 lb. per cu. ft. and per lin ft. The value of n is assumed at 15.

Mr. Turneaure. F. E. Turneaure, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Scott apparently bases his analysis and deductions on the assumption, as stated by him, that:

"The intensity of the tensile stress that can be developed in the steel at a cross-section is mainly dependent on this bond-friction-resistance."

Accordingly, he has assumed that, in the numerous tests quoted, the strength of the beams was in every case limited and determined by the strength of the bond, and, therefore, that the results obtained are a correct measure of such bond strength. A careful inspection of the original sources of information shows that, with very few exceptions, the strength of the beams tested was not determined by the bond strength, but by other factors, such as tensile strength of steel or strength of concrete. In most cases the cause of failure is indicated clearly by the results of the tests, or is pointed out distinctly by the investigator himself, and, with the exception of a few unimportant tests, the strength of the bond does not appear to have been concerned in any way with the results obtained, excepting that such strength of bond proved greater than the strength of the beam in other ways.

The author makes two principal deductions from his analysis: First, that the bond strength varies with the angle, α; and, second, that it varies largely with the relative amount of concrete between the rods. The data plotted in Figs. 4 and 5 constitute the main basis for the first conclusion, and those plotted in Fig. 6, that for the second conclusion.

Referring to these diagrams in detail, Fig. 5 gives the results of certain tests made at the University of Wisconsin. Consider, for example, the series G 1-G 4. In the bulletin in which these tests are described* it is distinctly stated that three out of four of these beams failed at or near the center as a tension failure; that is, by the over-stressing of the steel. The fourth beam failed by diagonal tension or shear. None of these failures was caused by slipping of the rods, and, in the first three cases, where the failures occurred at the center, it was very local in character, and wholly a question of tensile strength of steel and compressive strength of concrete. This is quite evident from the photographs of the beams shown on page 343 of the bulletin above referred to. The other series of tests shown in Fig. 5 represent

* Bulletin No. 148, University of Wisconsin, 1906, pp. 344 and 363.

failures in diagonal tension or shear; but in this series some slipping of the short bent-up rods occurred. The corresponding bond strength in such cases, however, is difficult to calculate, and is certainly not represented by the usual formula for bond stresses in horizontal bars. Mr. Turneaure.

In Fig. 4 the author deals in a similar way with tests made by Professor Talbot. As particular examples of the misuse of data in this figure, attention is called to the plotted values of Beams Nos. 50, 52, 53, 54, 55, and 56, of the series of 1906. All these beams failed in tension,* and at approximately equal stresses in the steel. Therefore, these results also give no information whatever concerning bond strength, excepting that the strength of bond was greater than needed in these cases. In these tests the position of the loads varied, and therefore the reaction required to develop a given bending moment necessarily varied, resulting in a corresponding variation in shear and bond stress, as indicated in the diagram. However, as the rods did not slip, such results can hardly be used as a basis for determining strength of bond.

It is difficult to understand how the author can reach deductions relative to bond strength from the tests which he has here quoted. In most cases the results of the tests would have been the same if the rods had been given an absolute and fixed anchorage at their ends.

With respect to the author's second deduction, the same misuse of data is evident. In Fig. 6 are plotted results of certain observations by the writer.† The results on Beams Nos. 8, 10, and 12 constitute one group, and those on Beams Nos. 5, 7, 9, and 11, a second group. The beams of the first group were practically identical, excepting that No. 8 was reinforced with four $\frac{3}{8}$-in. rods, and Nos. 10 and 12 with two $\frac{1}{2}$-in. rods, thus giving about the same percentage of reinforcement in both cases. "No trouble was experienced from the slipping of the rods except in one case where they had become displaced in the moulding."† The failures were caused either by the over-stressing of the steel or by a diagonal tension failure of the concrete. The three bars of the first group failed at almost exactly the same load, giving an average shearing stress of 130 lb. per sq. in., which may be considered a good value for beams reinforced with straight rods only. Because of the fact that in one case two rods were used, and in the other case four smaller rods, the calculated bond stress would be quite different, and, on the assumption of failure by slipping of rods, the author uses these results in Fig. 6. Similar conditions existed with respect to the other group of beams reported by the writer.

A still greater injustice has been done in Fig. 6 to the tests made

* Bulletin No. 14, University of Illinois Engineering Experiment Station, p. 15.

† *Proceedings*, American Society for Testing Materials, 1904.

‡ *Proceedings*, American Society for Testing Materials, 1904, p. 507.

Mr. Turneaure.

by Professor Howe.* Referring to Beams Nos. 1, 2, 3, and 4, which in Fig. 6 constitute an important group, these beams were all of constant breadth and constant percentage of reinforcement, but varied in depth from 4¼ to 12¾ in., and the number of rods varied from two to six, ½-in. rods being used in all cases. The results of the tests were very uniform, showing a calculated stress in the steel varying only between the limits of 74 300 and 76 500 lb. per sq. in. The elastic limit of the steel was about 60 000 lb. per sq. in. The results, therefore, are all that could be expected, from the standpoint of strength of steel. With respect to failures of the beams, Mr. Condron states:

"In most of the beams tested, fine hair cracks began to appear in the lower or tension side of the beams at about one-third of the maximum load, and the concrete crushed under the maximum load in 15 of the 18 beams. These cracks did not appear to any serious extent until nearly the maximum load was reached, when the stretching of the steel beyond its elastic limit permitted them to increase rapidly."

These, therefore, were purely tension failures, and gave no valuable information relative to bond strength. With the constant breadth of beam used and the variable number of rods, the results of the tests, when interpreted as bond failures, naturally must plot in the apparently significant line shown in Fig. 6.

With respect to the author's two main conclusions, it appears from these considerations that they are based on a misinterpretation of the results of the tests and, therefore, are wholly unwarranted, and of no significance whatever.

In other respects, the author seems to have fallen into the same error of misinterpretation. For example, he states in two or three places that deformed bars give little or no increase in bond strength, and quotes certain tests as a basis for this statement. Here, again, these tests are generally without significance in this relation.

With respect, also, to the value of stirrups, conclusions are drawn to the effect that as the proportion of concrete between the rods decreases, the value of the stirrups increases. In the tests quoted in that connection, the percentage of tension reinforcement varied from 0.81 to 1.62 per cent. In the case of the least amount of reinforcement, tension failures occurred, both with and without stirrups, and very little aid could be expected from them. In the case of the higher percentages of reinforcement, the beams without stirrups failed in diagonal tension at about the same loads, while those with stirrups were forced to fail in tension. The necessary result of these conditions was that the stirrups caused a greater increase in strength for the higher percentages of reinforcement, and the amount of concrete between rods does not appear to the writer to have any significance whatever in these tests.

* Reported by T. L. Condron, M. Am. Soc. C. E., in *Proceedings*, American Society for Testing Materials, 1904.

Mr. Turneaure.

Finally, the point at which slipping takes place seems to be determined by the author by a comparison of the elongation of the rods, as determined by extensometer measurements, with the calculated elongation determined on certain assumptions regarding the action of concrete and the position of the center of compression. Such calculations have been made in numerous instances in bulletins issued by the University of Wisconsin, and, on the whole, the calculated values agree with the observed value as closely as could be expected. Until the load has advanced well toward the ultimate, a close agreement cannot be expected. There is a good example of such calculations* in the case of 14 T-beams tested at the University of Wisconsin. The average stress in the steel, as determined by theoretical deduction, was 45 000 lb. per sq. in., while that determined by extensometer measurement on the edges of the T-beam flanges was 48 300 lb. per sq. in. In most individual cases the latter exceeded the former somewhat, which is contrary to the results as plotted by Mr. Scott. It would seem that the possible errors involved in such calculations would be too great to admit of any such deduction as is made by the author, unless the discrepancy was large and always of the same sign.

The writer regrets that he is unable to agree with Mr. Scott in his analysis and conclusions, but, from the best study which he has been able to give to the matter, it is his belief that the conclusions are unsubstantiated by the data, and, in fact, are wholly at variance therewith.

Mr. Scott.

William Fry Scott, Assoc. M. Am. Soc. C. E. (by letter).—In replying to the several discussions, the writer desires to acknowledge the generally kind and courteous treatment accorded his paper. He has examined these discussions with the greatest interest, and may be pardoned if he confesses to a feeling of no small satisfaction that a close reading has revealed to him no weak point in his theory—his position remains unshaken. One of the discussors criticizes the writer for not having "characterized as monstrous" certain errors that have "worked their way through the literature of reinforced concrete." To this he can reply that he deliberately refrained from mentioning fallacies, fancied or otherwise, throughout the entire paper. By dealing with the positive aspect of the subject, the negative side can be left to take care of itself. At any rate, the less said about mistaken theories the better.

Mr. Godfrey begins his discussion by saying:

"Although this paper does not give specific conclusions for practical designing, it could be made a basis for such conclusions."

The writer has looked in vain for what he had every reason to infer was promised in this quotation, namely, Mr. Godfrey's idea for

* Bulletin No. 197, University of Wisconsin, 1908, p. 99.

Mr. Scott.

this development of a basis for practical designing. The writer has also wondered whether Mr. Godfrey forgot about it as soon as the words flowed from his pen, or whether he "first endured, then pitied, then embraced" the specific conclusions of the paper. Mr. Godfrey differs only in degree from the theory quoted from Turneaure and Maurer, and he has apparently read the deductions of the paper hurriedly or he would not have assumed that they contained approval of that theory. If this paper stands for anything, it stands for the refutation of that theory.

Mr. Burroughs' discussion calls for a few words of explanation. The name "bond-friction-resistance" stands for an idea—the idea for which this paper was written—and this idea is diametrically opposed to the previous ideas of bond.

Mr. Maguire's contention is that the tensile strength of the concrete is preserved by the bond, notwithstanding Professor Turneaure's classic experiments to the contrary. If Mr. Maguire (Mr. Godfrey might also be included) were to count upon the tensile strength of concrete being preserved by the fictitious bond resistance, he would find himself in much the same predicament as the fabled dog that thought he saw reflected in the water a larger bone than the one he had in his mouth.

Mr. Goodrich,* in discussing this paper at the meeting at which it was presented, referred to the work of Professor Mörsch, which he had translated into English for publication elsewhere. This translation came out about the time the writer was gathering his notes for the paper, and therefore had not been reviewed by him. It contains a great deal of valuable data as further proof of the writer's theory, and he is therefore grateful for the material, some of which is reproduced herewith. It seems incomprehensible that, in his examination of the experimental data presented in this work, Mr. Goodrich did not see just such evidence as was presented in this paper.

At the same meeting Mr. Lindau* made the statement that the curve for actual elongation of steel for Beam No. 28, in Fig. 1, was shown too steep, and that it should be more nearly coincident with the curve for apparent elongation. As a matter of fact, this curve for actual elongation should be steeper by the difference between the mean value of the mass of material in the rod and the value based on the minimum cross-section on which the elongation was computed.

With regard to the discussions as a whole, it is manifest that they are rather more in the nature of vague questionings than seriously considered replies to the writer's theory of bond-friction-resistance. At any rate, most of the points brought forward had actually been

* E. P. Goodrich, M. Am. Soc. C. E., decided to withdraw his discussion, and A. E. Lindau, Assoc. M. Am. Soc. C. E., has not revised what he said at the meeting; it is therefore not printed here.—Editor.

considered by the writer before the presentation of the paper. Not only had these questionings been considered, but there were others which might be added to the list, the answers to which, however, can only be found by further experimental investigation. The questions are: Mr. Scott.

(1) How does bond-friction-resistance vary for values of cot. α greater than eight and less than two and a half?

(2) How does bond-friction-resistance vary for values of β greater than twice the sum of the perimeters of the rods?

(3) How does bond-friction-resistance vary for values of β less than two-thirds of the sum of the perimeters of the rods?

(4) What is the law of increase of bond-friction-resistance due to various methods of bending rods?

(5) What is the true value of stirrups for various conditions of intimacy between rod and stirrups?

(6) What is the effect of various bends under various conditions of tensile stress in the steel?

(7) What is the true value of concrete in compression due to flexure of reinforced concrete?

(8) What is the true shearing value of concrete in reinforced concrete beams?

Before considering the discussions in detail, a review of the general aspect of the subject, from a point of view slightly different from that taken in the paper, will be made.

In the shop and in the field the writer has observed that bent plates, the bend for which had been computed in the drafting room with logarithmic care, were as much as $\frac{1}{8}$ or $\frac{1}{4}$ in. out, and that rivet holes which had been fixed with nice precision did not match by $\frac{1}{8}$ in. or more. He has seen workmen measuring out materials for concrete, and mixing and watering them, in a manner that jarred his ideas of proportion, and gave a jolt to his conception of the ideal. The degree of perfection attainable, at any time past or present, is and can be only relative; the results of the laboratories represent only a slightly nearer approach to the ideal than is possible in the field, and even in the laboratory there are many approximations. A 6-in. slide-rule possesses a greater degree of refinement than is required for most of the problems with which one has to deal, simply because personal judgment must be brought to bear on elements which do not enter into equations and formulas. Moreover, this judgment usually modifies the results of those equations and formulas to a greater extent than does the lack of absolute accuracy in the slide-rule. The point the writer wishes to make by this reference to relative values in theory and practice, is this: The problems represented in the subject-matter of this paper were examined, not exclusively from the point of view

Mr. Scott.

of refined logarithmic calculation, nor exclusively from the point of view of the histologist, who concentrates his attention on a particle. They were examined, primarily from the viewpoint of the observer with field glasses, who surveys the operations of the entire field, using also, as occasion requires, precise methods for the investigation of points calling for special elucidation. The work done on the investigation of reinforced concrete in the past has been, figuratively speaking, for the most part microscopic; the telescope has been practically unused, and errors in deduction have resulted from concentrating too closely on the minutiæ of the problems to the exclusion of their settings and the interrelation of their parts. Thus, the paper was presented for the purpose of giving a combined telescopic and microscopic view of the subject.

The principal assumptions on which the mechanics of reinforced concrete are based have been founded on the work of inexperienced observers, who have failed to give complete and accurate reports to those who have used their observations for making deductions. The students, in whose hands the test programmes have been placed as thesis work, have not, as a general rule, had the necessary practical or laboratory experience to make either complete or accurate scientific observations on which to base deductions or to establish theories. That it is not possible for students to have such experience is shown by the fact that these same men are never available for the Profession until they have had two or more years in the annealing chamber of experience. That the universities make no pretense of turning out ready-made engineers is further evidence as to the mistake of expecting correct observations from students. Is there one student thesis in a thousand that would be anything more than a joke to the experienced engineer, or to its author five years after it was written?

By way of illustration of the fact that assumptions and theories have been based on the reported observations of students, the following is quoted from Professor Talbot. In commenting on the 1904 series of "Tests of Reinforced Concrete Beams," he observes:

"It should be said that the work of making these tests was done principally as thesis work, and that Messrs. * * * of the class of 1904, are entitled to credit for the care, thoroughness and untiring interest given to the investigation, and are to be commended for their skill and thoughtfulness."

Again, he says of the 1905 series:

"The work of testing the beams was done principally as thesis work. The data have now been worked over and the calculations checked, new drawings made, and results more fully studied and discussed."

He also says:

"As was stated in Bulletin No. 1, the principles governing the strength and action of reinforced concrete construction have not been

fully established, and the opinions and theories presented by engineers are somewhat conflicting. However, many of these points are being cleared up, and experimental work on reinforced concrete is gradually establishing the principles. The results given in Bulletin No. 1 have a bearing upon the action of reinforced concrete in simple flexure and upon the calculation of the strength of beams. The following topics and conclusions, among others, are there considered: the general action of the beam during the progress of the loading and the division of the phenomena included in the tests into four stages representing different conditions in the steel and concrete; the determination of deformations in the steel and concrete by careful instrumental work; the determination of the position of the neutral axis by experimental methods and the checking up of the tension developed in the steel by the use of the observed deformations and the position of the neutral axis thus obtained; an experimental determination of the amount of steel which will develop the full compressive strength of the concrete; for beams not having a sufficient amount of steel to develop the full compressive strength and not failing by secondary or web stresses, the conclusion that the maximum strength of the beam occurs at slightly above the yield point of the steel instead of the ultimate strength of the steel, and hence that the yield point of the steel used has an important bearing on the design of the beam; the conclusion that the form of the load-deformation curve goes to show that during the second stage the concrete fails in tension in such a way as to make this tension negligible in the calculation of the bending moment, instead of that the stretch is kept up in such a way as to continue to furnish strength to the beam, as has been contended."

Mr. Scott.

There are a number of claims made here to which the writer would like to refer, along the lines of his theories, but he will have to leave their consideration to the reader. The important reason for the quotation lies in the fact that Professor Talbot assumes that:

"In general the results were such as to give confidence in the value of experimental methods in establishing principles for use in the design of reinforced concrete construction if only the tests are made in a systematic and scientific manner."

Now, the writer finds that the regulation governing the conduct of thesis work in the University of Illinois is as follows:

"In all cases in which a thesis is required, the subject must be announced not later than the first Monday in November, and the completed thesis must be submitted to the dean of the proper college by June 1. The work must be done under the direction of the professor in whose department the subject belongs, and must be in the line of the course of study for which a degree is expected. The thesis must be presented upon regulation paper, and is deposited in the library of the University."

It is thus apparent that at the University of Illinois, as at other Schools of Applied Science and Engineering from which reports have emanated, the student is given this work, partly at least, to train his

Mr. Scott.

observation and to give him some opportunity to draw deductions, true or otherwise, from what he sees. However, tests are not necessarily made in a "systematic and scientific manner" merely because precise instruments are used or a university laboratory happens to be the place at which they are conducted. The fact, too, that the work is done under the general supervision of the professor, manifestly does not insure the accuracy and completeness of the observations; nor, *a priori,* the correctness of the inferences and the generalizations. The justice of the verdict of the court will be influenced in no small way by the correctness or falsity of the evidence submitted by the witnesses. Deductions made from faulty premises are invariably followed by troublesome complications. Clearly, no more stable basis on which to found a principle of design than that afforded by careful experiment can be desired, and equally, a more unsatisfactory one than that built up from inaccurate experimentation can scarcely be conceived. The effect of such is not far to seek. It is entirely the lack of "confidence in the value of experimental methods in establishing principles" which, on the part of the layman, has given value to the claims of the so-called "practical" man. Hence it is that Science has to shoulder the sins of incautious scientific investigators, in the same way that capable and experienced professional men must shoulder the sins of ignorance committed by others claiming to be in their ranks.

The literature of reinforced concrete has been flooded with elaborate reports, based mostly on observations made by students, giving the elongations of steel and of concrete in reinforced beams in most minute detail, and the deflections with microscopic care and precision. The plotted curves from these measurements are really very pretty—they are beautiful. The perfection and grace of their lines excite one's admiration to such an extent that their real values have not been properly investigated. To their hypnotic influence, as will be shown, may be attributed the loss of several years of valuable time in the progress of the science of reinforced concrete, to say nothing of the great waste of energy and material necessarily involved in that loss. Though the deformations shown on these curves were not perceptible to the naked eye, there was plenty of movement, only too plainly visible, that vitiated the deductions drawn. Unfortunately for the engineering world, the larger phenomena passed unobserved, the microscope having been used to the utter exclusion of the field glasses. The writer believes that many of these reported tests were not made in a thoroughly "systematic and scientific manner":

First, because the observations were left to students who had not become accustomed to the novelty of their instruments; and

Second, because some of the most vital and important phenomena were never even seen, to say nothing of being carefully recorded.

Mr. Scott.

While the glare of newness is on a student's problems, he is not capable of making complete and comprehensive observations on all that is going on before his eyes. This is precisely what has happened in the case of most of our reports of reinforced concrete experiments. The observers were under the spell of instruments such as "extensometers" and "micrometer mirrors," and could have no thought for the vitals of their problem. While they were recording extensometer readings and taking deflection observations with the marvelous precision of one part in a hundred thousand, the end supports of their specimens were separating from each other as much as one part in three or four hundred. As stated previously, the loss of several years of valuable time must be attributed to the deception perpetrated by these beautiful curves for elongation and deflection. The deception results from the neglect of the observers to take into account the above-mentioned separating of the supports. It is easily seen why this important phenomenon has been neglected. Perfect bond, that classic myth of two continents, has precluded the necessity for such observations, and the occurrence has generally passed unnoticed. In consequence, three important circumstances have been overlooked:

(1) The "actual elongation" of the steel over the entire length of the rod;
(2) The "apparent elongation" of the steel over the entire length of the rod; and
(3) The slipping of the rod.

In addition, the evidence brands these beautiful curves as deceptive. The effect of a pull applied at the end of a cord is to raise the horizontal tangent to its curve. This is what was taking place all the time those deflection observations were being made with microscopic precision. The thrust in the concrete, due to arch action, resulted in the spreading of the supports, and the consequent tension in the steel was partly counteracting the tendency of the specimen to deflect. These points will be discussed in more detail in what follows.

With regard to the related question of laboratory practice in general, the writer has observed that there is an idea generally prevalent that reinforced concrete specimens should be made to approximate, in their fabrication, the conditions obtaining in the field. Here is a quotation from a report on an important series of tests:

"In order that the concrete and steel should represent products likely to obtain in practice, Atlas cement, bank sand, crushed rock, and corrugated steel bars were purchased in the open market. The mixing was done by a local contractor of experience with his own gang of men and in the manner he ordinarily employed."

Now, while this method will doubtless give an idea of the ultimate strength of field work, the true aim of the laboratory experiment is

Mr. Scott.

rather to study the relation of the two materials to one another. To involve the problem needlessly by the introduction of an endless number of variables, due to careless workmanship and defective materials, is to defeat its purpose. The problem, therefore, should be attacked under conditions as nearly ideal as possible. A poorly mixed concrete is as different from well mixed material as cast iron is from steel. It is possible, by the observance of a few simple rules, to produce a uniform product which will be more or less constant in quality; and such rules should be laboratory rather than field precedents. The field man knows nothing about exact proportions of materials; the barrow is his standard of measurement, and he has no concern with the fact that one of his men has a barrow with an iron bucket of one shape and another with a wooden basket of another shape. The laboratory man should know how to measure materials by volume and check by weight, and he should know of and record a great many circumstances which will help to a true interpretation of the results of tests of specimens made under his supervision. Many tests have been made which would have been of untold value had the experimenters adopted true laboratory precedents, or what should have been true laboratory precedents, instead of attempting to approximate the conditions of the field.

When tests on reinforced concrete were first undertaken, the beams were made by simply embedding plain straight rods in the bottom of the concrete. The bond was the medium for developing the tensile strength of the steel, and the tensile strength of the steel was the medium for giving the requisite tensile strength to the combination. The principle underlying this combination of materials is an old one. It has its analogy in the principle of the simple truss, the upper chord of which is in direct compression and resists also the vertical shearing forces. Applying this principle to a reinforced concrete beam, it is observed that the vertical component of the diagonal tension forces is taken up by the vertical shearing strength of the concrete, while the horizontal component is taken up by the tensile strength of the rods. The force indicated by C', as in Fig. 22, may be assumed to be the analogue of the force acting on the compression member of a simple truss. It is thus seen that the only other force necessary to preserve equilibrium is that supplied by the tension on the steel. When the portion of concrete, 7-8-9-10, in Fig. 23, is substituted in the segment, 2-3-4, the bond resistance will consist, not only of the adhesion from 1 to 2, as in Fig. 22, but also the additional adhesion over the distance, 7 to 8. As shown by the writer, from "Some Tests of Bond of Steel Bars Embedded in Concrete by Three Methods," the value of the bond is of greater intensity over 1-2 than over 7-8; and when the entire segment, 2-3-4, is filled, as in Fig. 24, the value of the bond over 2-3 will be more or less of the same value as over 7-8. Now the spandrel,

2-3-4, in Fig. 24, is undoubtedly more or less valuable for assisting in carrying the vertical shearing stresses, and for giving some aid to the bond resistance in the rod, but, broadly speaking, as there is no diagonal tension in the simple truss, there will not be diagonal tension in the concrete of this spandrel if there is sufficient adhesion between the concrete and the steel to develop the tension necessary to reinforce the concrete as an arch in a simple truss. Mr. Scott.

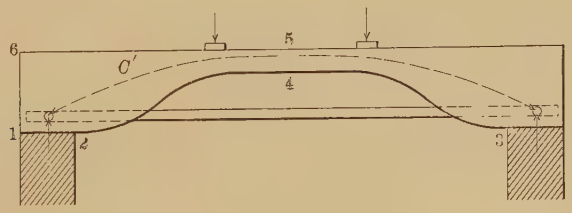


FIG. 22.

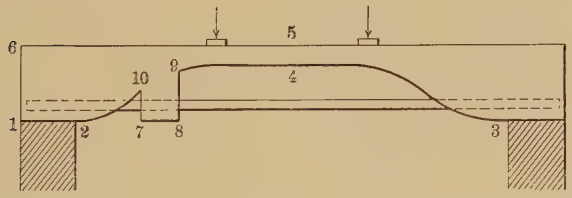


FIG. 23.

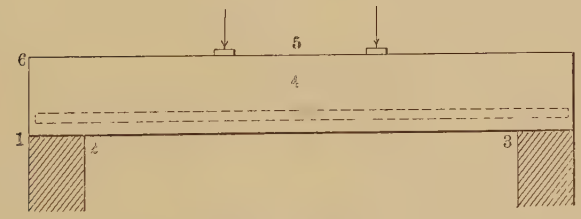


FIG. 24.

Professor Mörsch says, of the theory of reinforced concrete:

"In the early stages of the development of reinforced concrete, its builders had at hand no recognized method of calculating, and Monier and François Coignet erected their work solely by practical instinct and experience. * * * Practice has * * * been far ahead of theory. The principal question in controversy has been whether the tensile strength of the concrete in bending should be considered. Among practical builders this question was really decided at the start, and

Mr. Scott. decided against its inclusion, because absolutely no attention is paid to it and the steel is stressed to the maximum safe limit. The tensile strength of the concrete is entirely ignored."

Under the heading, "Failure of Bond Between Steel and Concrete,"* Professor Talbot says:

"Primary failure by the breaking of the bond between steel and concrete is unusual for beams having the proportions of ordinary test beams. The bond stress developed in such beams at their ultimate load * * * ranges from 70 to 193 lb. per sq. in., and the bond tests on plain mild steel rods give values from 200 to 500 lb. per sq. in. and on some forms of deformed bars from 300 to 1 000 lb. per sq. in. * * * In most of the failures reported to be caused by slipping of the bars, it seems certain that this slipping occurred subsequent to diagonal tension failures or other changes which were the primary causes of failure. For mild steel reinforcement placed horizontally in beams of ordinary dimensions, the diagonal tensile strength of the beam will be a much weaker element than the bond stress between steel and concrete."

Here he attributes primary failure to tension in concrete, in spite of the fact that tension is "entirely ignored," as pointed out by Professor Mörsch in the preceding quotation; and, further, he wholly ignores the fact that a perfect bond would not permit the diagonal tension to develop. Suppose these anomalies to be ignored, and that the case be examined as presented by Professor Talbot: In any study of the stresses and strains in reinforced concrete, only like things should be compared. This is a somewhat old and time-worn truth, but, the fact is, it seems to have been practically overlooked in the paragraph just quoted from Professor Talbot, which may be said to be generally approved in the consideration of this particular question. For instance, the bond strength referred to was studied from pull-out tests, where the concrete crowded around the rod in a state of compression, and the values thus obtained were accepted as evidence that the bond strength on steel embedded in beams would be at least equivalent. This evidence indicated that shorter rods than were usually embedded in beams would have their ultimate strength developed by the bond, and the question was therefore accepted as settled finally and conclusively that adhesion was perfect and would produce a practically homogeneous beam. Now, if the original evidence had been based on tests of like conditions which exist in beams, such an assumption would have been justified, but it is not justified under the circumstances. Two unlike things have been compared. The bond on the rods embedded in beams has a normal component partly in tension, and not entirely in compression, as in pull-out tests. That this fallacy has worked its way into the very foundation of present-day theory may be seen by a review of

* "Tests of Reinforced Concrete Beams, Series of 1905." Bulletin No. 4, University of Illinois Engineering Experiment Station.

the assumptions recommended as "a basis for calculations relating to the strength of structures," by the Joint Committee on Concrete and Reinforced Concrete. These assumptions are: Mr. Scott.

(*A*) A perfect adhesion exists between concrete and reinforcement;
(*B*) A plane section before bending remains plane after bending;
(*C*) The tensile stresses in the concrete shall be neglected.

Assumption *A* amounts to this: that "the internal stresses and strains in the concrete are to be determined upon the assumption of its being a homogeneous material," as stated by Professor Mörsch.* It is difficult, however, to see how two materials with such entirely different characteristics as concrete and steel can produce a combination that will possess, even as far as practical results are concerned, the mechanical properties attained by either one when acting independently. It is clear that they do form a combination which gives really wonderful results, but it is asking too much to believe blindly that they become homogeneous in effect. Assumptions and theories do not prove the point. This question was studied by the writer, as reported in the paper. He also studied it experimentally in the Engineering Laboratory of the University of Toronto, and, as his experiments were planned in the light of the preceding investigation, they will be appreciated for their results. These results will be reviewed, after considering the question of the appearance of minute cracks in reinforced concrete.

It would seem reasonable to expect that the presence of cracks, visible or otherwise, would depend to a great extent on the degree of perfection of the bond. If the adhesion were perfect, as assumed by the Joint Committee, the cracks would be infinitesimal in size, and there would be a myriad of them; whereas, if the adhesion were more or less imperfect, slipping would occur, and cracks would represent an accumulation of smaller ones, and would then be of sufficient size to be visible. In other words, the occurrence of visible cracks in a reinforced concrete beam is conclusive proof that slipping has occurred, and that the bond has not been perfect. Attention, therefore, is directed to the following corollaries:

(1) According to Assumption *A*, tension failure cracks should be minute and infinitesimally close together;

(2) According to Assumption *A*, the first cracks should appear where the maximum stress in the steel exists;

(3) According to Assumption *A*, the first crack should widen as the stress on the steel becomes greater;

(4) According to Assumption *A*, the first crack should always be the most prominent one;

(5) According to Assumption *A*, there should be no cracks near the support, for either concentrated or uniform loading, nor should

* Appendix, "Concrete-Steel Construction."

Mr. Scott.

cracks be apparent anywhere in the concrete unless the stress in the reinforcement is materially in excess of 3 000 to 6 000 lb. per sq. in.;

(6) According to Assumption *A*, the last crack will be nearer the support than any previous crack, but it should be small compared with the first;

(7) According to Assumption *A*, the last crack will be somewhat nearer the support for a uniform load than for a concentrated load, other things being equal;

(8) According to Assumption *A*, the size of the cracks should vary with the intensity of the stress in the steel.

The following is quoted from "Tests on Reinforced Concrete Beams,"* by E. A. Moritz, Assoc. M. Am. Soc. C. E.:

"The appearance of the first minute cracks in reinforced concrete beams under stress has been the subject of some controversy among investigators. According to tests made by Considère in France the first cracks on the tension side of a beam do not occur until the extension is about ten times as great as concrete of the same quality, unreinforced, will stand. Considère makes use of this supposed phenomenon in his formulas by assuming that the concrete carries part of the tension up to the safe working stress of the steel.

"To determine if possible the reliability of the above results, a test was made similar to Considère's. The system of loading used gave a uniform bending moment over the middle 8 inches of the beam. Considère subjected his beams to a uniform bending moment over the entire beam. In order to note the cracks more readily, the beams were tested with the tension side up. It had been previously observed by Mr. Hartman that during the first stages of the test wet streaks occurred on the tension face of the beam, and by further observation it was noticed that these wet streaks afterward developed into cracks. This gave a convenient means of finding when the first crack occurs. These wet streaks or 'water marks' as they will be called, were found to occur only in beams which have been stored in water and thus contained considerable moisture. The beams were removed from the water several hours before testing so that although when tested the surface was dry considerable free moisture was contained in the body of the beam. The probable explanation of the appearance of water marks is that, when a very minute crack occurs on the surface and gradually extends into the body of the beam, the water is brought to the surface by capillary attraction.

"To determine the truth of the assumption that the water marks were actually minute cracks, a special test was made on a beam. * * * This beam was 3″ x 6″ in section, 50″ long (48″ span), reinforced with one half-inch Ransome twisted bar, was loaded with two concentrated loads 4″ on either side of the center, and was tested at the age of six months. The load was applied slowly until several water marks had developed across the face of the beam. No crack was visible to the naked eye, but with the aid of a microscope a crack about $\frac{1}{4}$″ long could be seen at one of the marks. The

* Bulletin No. 148, University of Wisconsin.

load was now removed and by means of an emery saw a slice of the beam between the steel and tension face was removed. When the saw reached the first water mark the piece fell out, and as each mark was reached another piece fell out, showing that cracks existed at these points. These slabs were tested in bending and found to be perfectly solid between marks. The photographs, Plates N-I and N-II [Figs. 1 and 2, Plate XVI], show the positions of the water marks and the detached slabs. By comparing the two photographs, it is seen that the positions of the breaks in the slabs and the positions of the large water marks coincide. Mr. Scott.

"It is thus evident that the concrete cracks much sooner than can be detected with the eye, and having once cracked, no matter how minute the crack may be, its strength in tension is zero. In later tests these water marks were found to occur when the stress in the steel was as low as 3,000 pounds per square inch. Tables N-1, K-3 and K-4 [not reproduced herewith] give detailed information regarding the time of appearance of the water marks. Variations in this respect among the different beams may be due to the fact that some of the beams, when tested, were wetter than others but it is to be expected that in work of this character the accidental variations will be considerable. For structural purposes the working strength of mild steel is usually taken as from 12,000 to 16,000 pounds per square inch; consequently it is plain that the concrete has lost practically all its tensile resistance long before the steel is stressed up to its working load, and therefore cannot be counted on to take any part of the tension at that load.

"Comparative tests of reinforced and plain concrete beams showed that the first water marks appeared on the reinforced beam at about the same load at which the plain beam failed. No water marks were observed on the plain beams, as these beams failed suddenly when the ultimate tensile strength of the concrete was reached. In the reinforced beams the concrete had stretched considerably more when the first *visible crack* was observed than had the plain beam, at rupture; but it will be found that the first appearance of the water marks, which are really minute cracks, coincides closely with the point of rupture of the plain beam. The elongation of the concrete at the time when the first water mark was observed varied between 0.0001 and 0.0002 in. corresponding to an average stress in the steel of about 3,000 to 6,000 pounds per square inch. The appearance of the first minute cracks seems to be indicated also on the stress-strain curves. It will be noticed that most of these curves have a sharp bend at the point corresponding closely to the beginning of these cracks, indicating a change in the law of deformation at this point."

And further:

"In connection with the position of the neutral axis the main points discussed are the position of the axis obtained from tests as ordinarily made; the position as found in beams with the rods exposed so as to be unaffected by the surrounding concrete; the movement of the axis; and a comparison of the results with theory.

"Messrs. Gilman and Kahn made their tests on beams arranged in the usual way, that is, with the rods completely surrounded by concrete. * * *

Mr. Scott.

"The stresses in the steel * * * will be seen to be * * * much higher than would be deduced from the observed elongations of the steel, using the usual modulus of elasticity. Thus in the case of beam No. 12 * * * the elongation at rupture was 0.00115. Assuming a value of E equal to 29,000,000 the stress would be 33,000 pounds per square inch, whereas the calculations from actual moment give a stress of 53,000 pounds per square inch. This discrepancy is doubtless due partly to the initial tension which probably existed in the rods, as the beams were hardened under water. This tension, of course would not be indicated in the extensometer readings. That there was such a large initial tension seems to be shown by later tests on beams hardened in air, in which the agreement between the two calculations was much closer. The remainder of this discrepancy as will be shown is probably due to the fact that the elongations as obtained above do not represent the actual stretch of the steel at every point but merely show the average elongations; that is, the steel stretches much more, and is consequently stressed higher at the points where the concrete cracks, but these large local elongations are not shown in the extensometer readings. The reason for this is that the gripping of the concrete around the steel prevents it from stretching freely excepting at the points where the concrete cracks. Where there are no cracks, the concrete helps to carry the stress; and where there are cracks, the stress is carried entirely by the steel.

"As the extensions as obtained above are only the average, the position of the neutral axis as obtained from these extensions would be only the average position, its position where cracks occur being higher than over other portions of the beam.

"TABLE M—1. [TABLE 6.]

"*Comparison of stresses calculated from extension and from moments.*

"Load.	"Beam C 2.		"Beam C 3.		"Beam C 4.		"Beam C 1.		
	Sm.	Se.	Sm.	Se.	Sm.	Se.	Load.	Sm.	Se.
900	7 400	8 500	7 500	5 800	7 500	7 000	2 100	17 800	15 600
1 200	9 850	11 600	10 100	8 900	10 000	9 900	2 400	20 300	18 200
1 600	13 250	15 200	13 500	13 000	13 300	14 000	2 700	22 800	21 000
2 000	16 600	19 300	16 800	17 200	16 700	17 600	3 000	25 400	22 900
2 400	19 900	23 500	20 100	21 500	20 000	23 500	3 300	27 900	26 600
2 800	23 300	28 300	23 500	24 700	23 300	26 400	3 600	30 400	30 000
3 200	26 700	32 100	26 900	28 700	26 700	30 900	3 900	32 900	32 900
3 600	30 200	36 600	30 200	33 000	30 000	33 900	4 200	35 500	36 600
4 000	33 600	40 400	33 600	37 200	33 400	36 800	4 500	38 000	39 100

"Sm. is the stress in the steel as computed from bending moments.

"Se. is the stress in the steel as computed from measured extensions."

"To study this point more fully beams were made with the steel exposed over the middle portion of the beam on which extensometer readings were taken. By this means it was hoped to obtain the actual position of the neutral axis and stretch in the steel unaffected by surrounding concrete. The first test of this kind was made on four 5″ x 3″ x 48″ mortar beams containing four ¼-inch round rods and having the steel exposed over the middle sixteen inches (see Plate M-I) [Fig. 1, Plate XVII]. The beams were subjected to a uniform

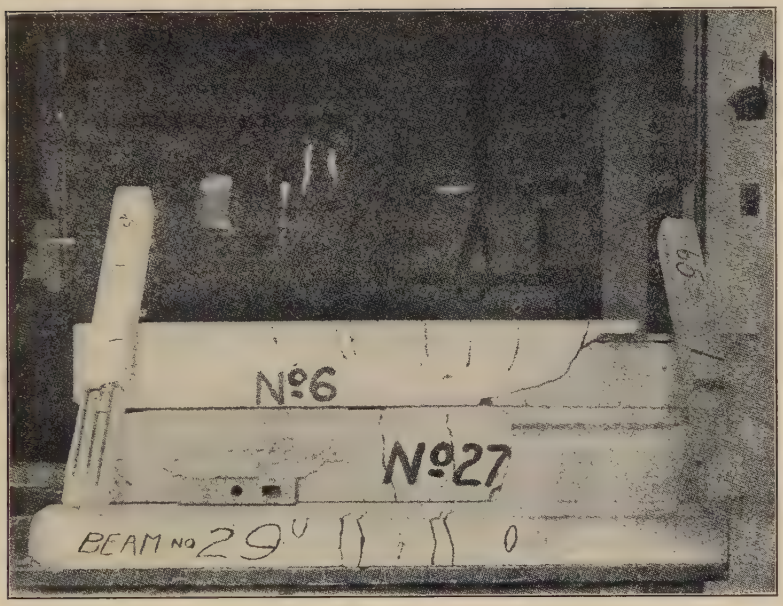


FIG. 1.

FIG. 2.

bending moment over the portion covered by the extensometer readings. The stresses in the steel were calculated from the bending moments on the assumption of a straight line variation of stress and also from the actual extensions. The results of these calculations are given in Table M-1 [Table 6]. Reference to the table will show that the agreement between the two stresses is quite close, much closer than has been found for beams with rods covered, an indication that the concrete surrounding the rods in beams as ordinarily made has a considerable influence on the strength of the steel. It thus influences the calculated position of the neutral axis, the tendency being to give a position nearer the steel than is actually the case." Mr. Scott.

Thus it is found that the first minute cracks appear when the steel is stressed less than 6 000 lb. per sq. in., and it is important to notice that they occur several inches apart, with the concrete intact between them. This evidence alone proves the fallacy of perfection of bond. Further, if "concrete cracks much sooner than can be detected with the eye, and having once cracked, no matter how minute the crack may be, its strength in tension is zero," why does that concrete still have strength in diagonal tension? The suggestion is absurd. Equally absurd is the almost superstitious belief that:

"The concrete surrounding the rods in beams as ordinarily made has a considerable influence on the strength of the steel. It thus influences the calculated position of the neutral axis, the tendency being to give a position nearer the steel than is actually the case."

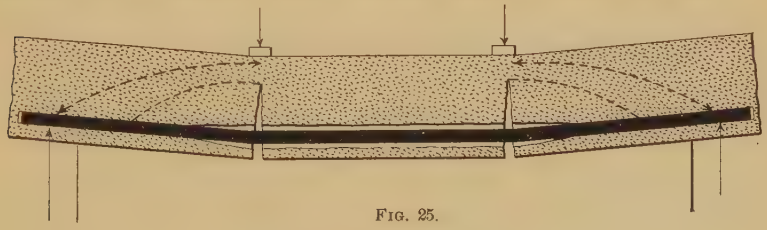


FIG. 25.

FIG. 26.

Such beliefs as this are due to a slavish faith in Assumption *A*. In the case of Beam No. 12 referred to, the writer believes that there is one and only one explanation, namely, that the portion of the concrete between the load points, and to which the extensometers were attached, remained more or less intact as a voussoir of an arch. The line of thrust is somewhat above the middle third; the bearings are outside of the vertical planes of extensometer attachment, and the rod has slipped through its casing as if a hole had been made for it. Hence this apparent elongation, as determined by the extenso-

Mr. Scott.

meter, is not of the steel, and is due to the eccentricity of the line of thrust on the concrete voussoir.

Fig. 25 is a diagrammatic representation (slightly exaggerated) of such a beam under stress. Consider the central block of the beam under the compressive stresses which would act on it, and imagine that there is no contact with the rod between the load points. Such a condition would be as illustrated in Fig. 26, where the specimen is really an eccentrically loaded column. It is evident that the curvature produced on this central portion by the eccentricity of the arch thrust on the voussoir will vary according to the accepted laws of stress and strain. If the rod were free to move over this portion, the curvature would be entirely due to the compressive stresses, C_1 and C_2. If it were not free to move, there would be a change in the curve from that resulting from the stresses arising from C_1 and C_2 only. However, it is not the writer's intention to go into a detailed analysis of the theoretical aspect of such questions, as they are secondary, and do not bear in a vital way on the theory he has proposed. It should be mentioned, however, that in actual beam tests the central block in Fig. 25 has usually one or more divisions; and, as the number of divisions multiplies, the nature of the curve becomes more and more complex. Further, if extensometers had been used over this portion of the beam, the results given by their readings would have been more or less erratic, according to whether or not the block had one or more voussoir divisions, and also according to the influence of the bond and the slip of the rod.

A careful examination of cracks in experimental beams* will show that:

(1) First cracks, for actual tension failure of the concrete, occur near the point where maximum tension exists.

(2) These first cracks may or may not become bond failure cracks, the tendency generally being for them to close up as the true bond failure cracks develop, because the tendency of the beam is to break into voussoirs, due to eccentricity of thrust, as previously shown.

(3) The first vital crack always appears at the cross-section having the maximum bond stress, and not at the cross-section where the maximum tensile stress occurs in the steel. This fact in itself throws a flood of doubt on the question of perfection of bond.

(4) For concentrated loads, the first vital crack occurs somewhere within the base of an equilateral pyramid having its apex at the maximum load point.

(5) For uniform loading, the first vital crack occurs somewhere in the neighborhood of the first quarter point from the reaction.

(6) Vital or bond failure cracks become the most prominent ones, while concrete tension failure cracks tend to some extent to close up.

*"Concrete-Steel Construction," by Professor Mörsch, Figs. 131-136, 152, 156, 157, and 160.

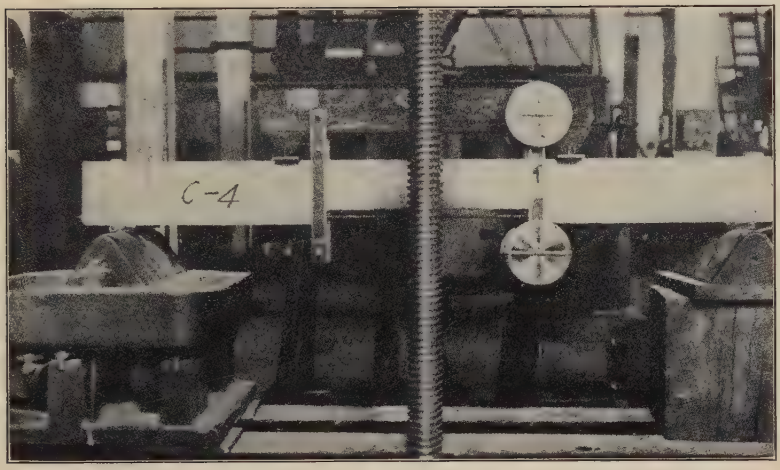


FIG. 1.

FIG. 2.

Mr. Scott.

If slipping be ever so slight, there is a partial shirking of its function by the rod, resulting in a portion of the tension being thrown back on the concrete. Where this occurs, diagonal tension may follow as a secondary failure, the primary one being a break-down of the bond. The elongation of the steel, after the tensile strength of the concrete has been exceeded, only partly accounts for the presence of hair cracks. Even when the elastic limit of the reinforcement is passed, the cracks are not found in the region where the stress in the steel is a maximum, and indeed, often occur where it is less than what would be required to produce tension failure in the concrete. It is thus seen that tension cracks in concrete may develop into bond failure cracks, which localize the indications for the slip of the rods, and these in turn into ultimate failure cracks. Thus one crack may represent three or more kinds of failure at different stages in the loading.

Another effect of the slipping of rods is to destroy the molecular arrangements of the concrete on the compression side of the beam, tending to throw the burden more and more on the extreme fibers and at the same time to relieve the others. However, concrete is particularly adapted to meet such a contingency. The truth of this will be seen from a study of the modulus of elasticity of concrete in compression. The deflection of the curve shows that, under high stresses, the material becomes semi-plastic. A vertical crack starting at the bottom of a beam either separates or elongates every fiber from the bottom to the top in proportion to its distance from the advancing edge of the crack. As the crack widens, the work of the extreme compressed fiber increases also. This, however, is relieved to some extent by the plasticity above referred to. The writer believes that there is a partial yielding of the walls of the cells or voids of the mass, and that this yielding results in the adjacent fibers assuming some portion of the load. This accumulative adjustment operates to render the compressive stress over the upper portion of the section nearly constant, and to supply the necessary compressive resistance until the ultimate strength of the beam in other ways is reached. It is believed, also, that very few of the so-called compression failures in the recorded experiments which have been reviewed in the preparation of this paper have really been properly designated. In nearly every case, the rods have first slipped and then, because of the crack, which is either vertical or diagonal, the divided parts have tipped up on their top edges, and then spalling inevitably began. This is not compression failure—it is bond failure primarily, as may be seen from Professor Mörsch's Fig. 86.* It is believed also that if the rods are kept from slipping, very few beams will fail primarily through compression.

* "Concrete-Steel Construction," p. 91.

Mr. Scott.

Final cracks sometimes occur as diagonal cracks extending more or less from the support toward the load point. These are usually called diagonal tension failure cracks, and a great many failures are attributed to them as the primary cause. Too much consideration has been given to diagonal tension in reinforced concrete beams, for the very reason that the fundamental idea of combining steel with concrete is to supply tensile strength where little exists. Now, if the function of the steel is to reinforce for tension, why make excuses for its failure to do so by the feeble device of variety in name? Professor Talbot has called attention to the distinction between primary and secondary failure, and says:*

"In all tests involving a determination of the cause of failure, care should be taken to distinguish primary failure from secondary or ultimate failure. While such distinction can not always be made, the value of the conclusions must depend upon the accuracy of the information on this point."

It is remarkable that Professor Talbot should have missed so completely the very point concerning which he cautions his readers. As an examination does show, the failure of the bond is the primary cause of break-down in every one of the beams, the tests of which are recorded in Bulletin No. 1.

As to the effect of this slipping on the strength of reinforced concrete beams, it is stated in the paper that: "With regard to the determination of the neutral axis * * * the ordinary methods * * * may be used for the working stresses." The reason for this is the belief that slipping does not begin much before three-eighths of the bond-friction-resistance is attained. This would carry the loading up to the working stresses that usually govern in design. On the other hand, the effect of slipping at the ultimate strength is to increase the length of the lever arm probably from 10 to 20% of the computed length. This increase usually occurs after the maximum load point has been reached, and it develops with the drop in the load. At or near the maximum load, it probably seldom exceeds 5 or 6% of the computed length of the arm of the couple.

To elucidate some of the points discussed in the paper, the writer conducted a series of experiments. A great deal of thought was given to the design of the experimental beams, with a view to developing true comparative results, free from foreign influences. The dimensions of these test beams were fixed in the following way: In all beams with like section moments but variable spans, shearing stresses vary inversely as the span; therefore, it was believed that short-span test beams would develop weaknesses in reinforced concrete that long-span beams would not exhibit. It was further believed that, if As-

* "Tests of Reinforced Concrete Beams, Series of 1905." Bulletin No. 4, University of Illinois Engineering Experiment Station.

sumption *A* were true, long-span beams could develop no weaknesses that would not be developed earlier in short-span beams. This will be understood from a study of the diagrams on Fig. 27. Mr. Scott.

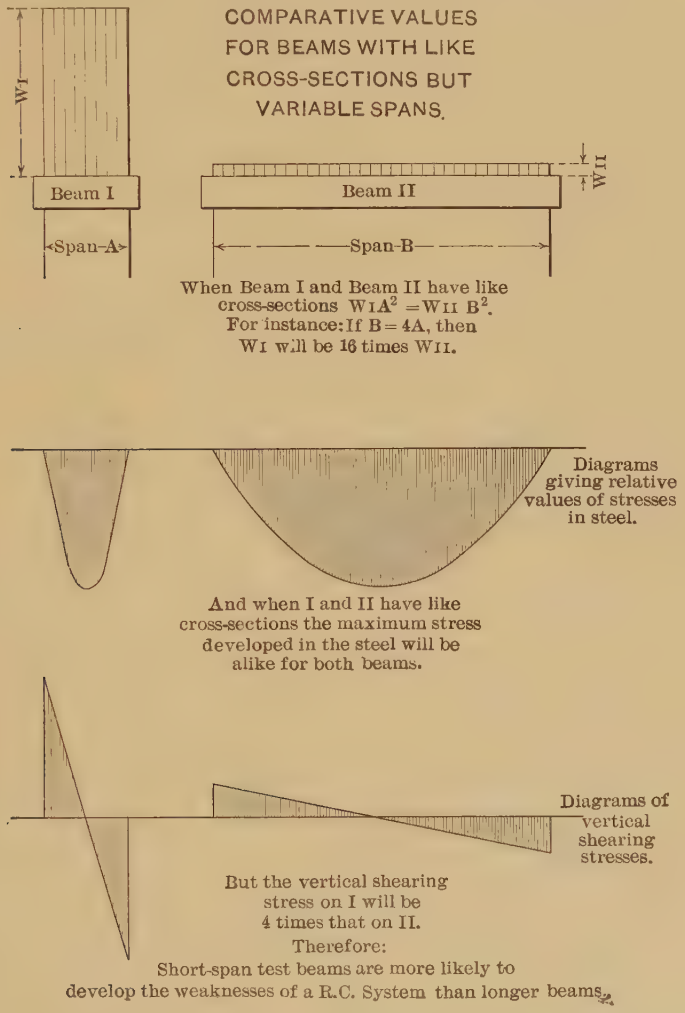

COMPARATIVE VALUES
FOR BEAMS WITH LIKE
CROSS-SECTIONS BUT
VARIABLE SPANS.
W I
Beam I
Span A
W II
Beam II
Span B
When Beam I and Beam II have like cross-sections W I A² = W II B².
For instance: If B = 4A, then W I will be 16 times W II.
Diagrams giving relative values of stresses in steel.
And when I and II have like cross-sections the maximum stress developed in the steel will be alike for both beams.
Diagrams of vertical shearing stresses.
But the vertical shearing stress on I will be 4 times that on II.
Therefore:
Short-span test beams are more likely to develop the weaknesses of a R.C. System than longer beams.

FIG. 27.

The beams, Fig. 28, were made as wide as could be put between the screws on a 100-ton Riehlé testing machine, namely, 18 in., and the percentage of steel was fixed so as to keep the maximum stress on the extreme fibers of the concrete within its ultimate compressive strength when the steel would be stressed in tension to 100 000 lb.

Mr. Scott.

per sq. in. The forms were made for a splayed T-beam, as shown in the isometric view, Fig. 28. An effort was made to embed the steel in the specimens so that the center would be 10 in. from the top of the beam, thus giving an effective net cross-sectional area of 180 sq. in. The actual location varied somewhat, but care was always taken to measure the exact position before destruction had rendered

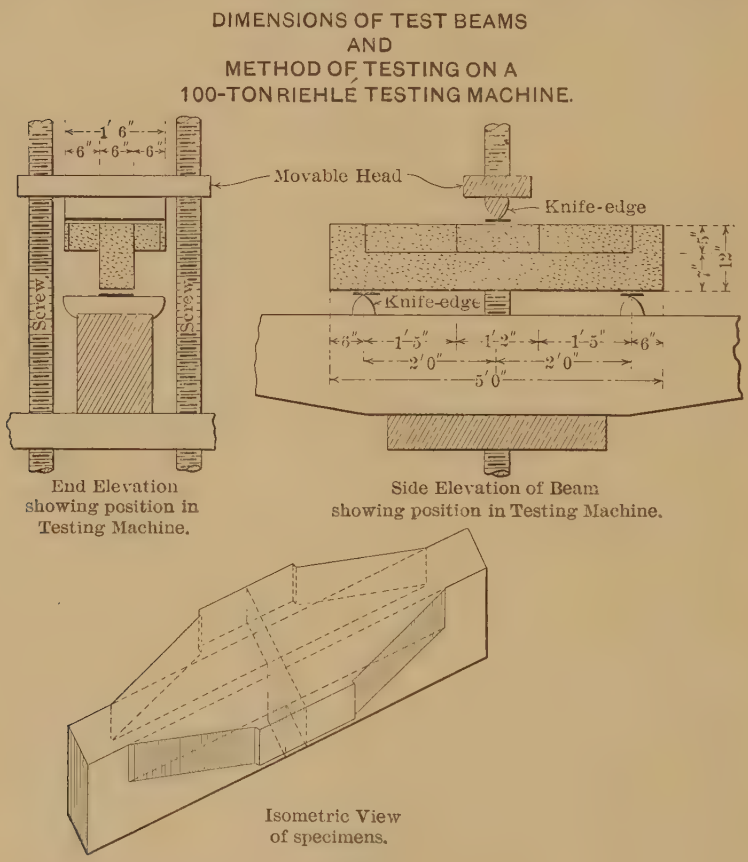


FIG. 28.

this impossible. The exact values are given in Table 7. Two sizes of rods were used: (*a*) plain round rods $\frac{5}{8}$ in. in diameter, with a nominal area of 0.3 sq. in. and a perimeter of nearly 2 in., the ratio of perimeter to cross-section being 1 : 6.4; and (*b*) twisted bars $\frac{1}{2}$ in. square, with a nominal area of 0.25 sq. in. and a perimeter of 2 in., the ratio of perimeter to cross-section being 1 : 8.0. The $\frac{1}{2}$-in. rods thus had 25% more area exposed to bond per square inch of cross-

TABLE 7.

Consecutive number...	145	146	147	148	149	150	151	152	153	154	155
Series number.....	26	14	19	24	12	23	16	25	15	18	13
Description.....	One-piece stirrups and bar.	All straight bars.	Anchored.	Ends heated, anchored.	Anchored.	All straight rods.	All straight rods.	All straight rods.	All straight rods.	Anchored.	Anchored.
Broken stone, in pounds	253.8	272.5	269.0	265.3	268.8	263.8	274.3	261.25	269.5	270.5	267.3
Sand, in pounds.....	168.5	153.3	149.8	169.3	163.3	168.3	151.0	171.25	153.75	150.0	162.0
Cement, in pounds.....	79.0	79.0	79.0	79.0	79.0	79.0	79.5	79.0	79.0	79.0	79.7
Percentage of cement..	18.7	18.5	18.8	18.2	18.3	18.3	18.7	18.3	18.5	18.8	18.6
Water, in pounds.....	47.0	44.0	44.0	47.0	47.0	47.0	44.0	.....	43.0	44.0	44.0
Kind of steel.....	.....	Twisted	Twisted	Twisted	Twisted	Medium	Medium	Hard	Hard	Hard	Hard
Nominal diameter, inches.....	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{5}{8}$	$\frac{5}{8}$
Section.....	Square	Square	Square	Square	Square	Round	Round	Round	Round	Round	Round
Number of rods.....	1	2	2	2	2	6	2	2	2	2	2
Diameter of rods.....	.....	.....	....	.. ..	.....	.....	0.622 0.629	0.627 0.616	0.616 0.610	.....	0.615 0.622
						.....	0.622 0.630	0.624 0.618	0.621 0.623	.....	0.620 0.618
Average diameter.....	.....	.....	.....	.....	.....	0.6235	0.626	0.621	0.6175	0.621	0.618
Area of steel, in square inches.....	0.599	0.5	0.5	0.5	0.5	1.83	0.614	0.605	0.597	0.605	0.595
Depth to center of steel, in inches.....	10.49	11.19	10.25	10.38	10.625	9.686	10.32	10.29	10.56	10.0	10.06
$B \div \Sigma O$.....	2.0	1.5	1.5	1.5	1.5	0.5	1.5	1.5	1.5	1.5	1.5
Cot. α.....	2.3	2.1	2.3	2.3	2.2	2.5	2.3	2.3	2.3	2.4	2.4
Made.....	Aug. 5, 6 P. M.	Jy. 24, 1 P. M.	Jy. 29, 5 P. M.	Aug. 5, 5 P. M.	Jy. 24, 11 A. M.	Aug. 5, 4 P. M.	Jy. 29, 3 P. M.	Aug. 5, 5 P. M.	Jy. 24, 1 P. M.	Jy. 29, 4 P. M.	Jy. 24, 12 M.
Tested.....	Sep. 4, 4 P. M.	Aug. 24, 5 P. M.	Aug. 28, 4 P. M.	Sep. 6, 7 A. M.	Aug. 24, 12 M.	Sep. 6, 10 A. M.	Aug. 28, 3 P. M.	Sep. 4, 12 M.	Aug. 25, 4 P. M.	Aug. 30, 5 P. M.	Aug. 25, 12 M.
Weight, in pounds.....	546	.....	.....	.....	.....	547.5	.....	548.5	.....	.....	.....
Maximum load, in pounds.....	27 650	26 800	39 100	46 400	50 800	23 000	19 480	20 250	20 000	46 950	45 400

Mr. Scott.

section of steel than the $\frac{5}{8}$-in. rods. Two rods were placed in each beam. Hard and medium steels were used for the plain rods; the surface of the hard steel was very much like that of the medium steel, as far as the eye could detect. The hard steel had an ultimate tensile strength of more than 100 000 lb. per sq. in., and the medium steel about 60 000 lb. per sq. in. The elastic limit was probably 55 000 lb. for the hard steel and 30 000 lb. for the medium. The twisted bars had an ultimate strength greatly in excess of 85 000 lb. per sq. in. When twisted rods were used, the nominal percentage of steel was $\frac{28}{100}$ of 1%, and when plain rods were used $\frac{34}{100}$ of 1%—approximately $\frac{1}{4}$ and $\frac{1}{3}$ of 1%, respectively. Great care was used in proportioning and mixing the materials. The stone and sand were proportioned by volume and also weighed, and the cement and water were weighed, 100 lb. of cement being considered as equivalent to 1 cu. ft. The specimens were 5 ft. long, over all, and were centrally loaded on a span of 4 ft. from center to center of knife-edges. Thus, according to the mechanics of the beam, the stresses have the following relation to the loads producing them: The stress in the steel, in pounds per square inch, is numerically equal to 2.17 times the load for the $\frac{5}{8}$-in. rods and 2.6 times the load for the $\frac{1}{2}$-in. rods. The stress in the concrete is equal to $\frac{1}{40}$ of the stress in the $\frac{5}{8}$-in. rods and $\frac{1}{44}$ of the stress in the $\frac{1}{2}$-in. rods. The bond-friction-resistance is equal to $\frac{1}{17\frac{1}{4}}$ of the stress in the $\frac{5}{8}$-in. rods, and $\frac{1}{21\frac{1}{6}}$ of the stress in the $\frac{1}{2}$-in. rods. It was noticed that failure usually occurred about 2 in. to one side of the transverse center line of the beam, and it was therefore assumed that the 4-in. plate which transmitted the load from the knife-edge to the beam did so through its two edges, thus giving the equivalent of two loads 4 in. apart. This would cause a stress on the steel equal to 92% of the values given.

The details of these beams are given in Table 7, and the comparative results in Table 8, Plates XVIII, XIX, and XX. Beams Nos. 16, 25, and 15 of this series were made as nearly identical as possible in every way except that the elastic limit of the steel in No. 16 was about 30 000 lb. per sq. in., while that of the steel in the others was about 55 000 lb. per sq. in. These three beams failed with positively characteristic bond failure cracks, and there was practically no difference in bond value, whether the steel was of high or low elastic limit. A marked distinction, however, was evident in the character of the failure after the maximum load was passed, which was plainly attributable to the relative elastic properties of the steels. This phenomenon is illustrated by Fig. 2, Plate XVII, and Plates XXI and XXII. All three beams developed the first crack under the load point, and the maximum load was passed before there was any apparent difference in behavior. The curves in the diagram, Fig. 29, give the observed deflections for these beams, and in every case the maximum load was attained at a deflection of about $\frac{1}{20}$ in. in

(1)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)
Number. $\frac{p_l f_b}{s}$	Maximum Load.	M_B Foot-pounds	$f_s = \frac{M_B}{A_s d'}$	Percentage of Standard B-F-R.	$\frac{f_s}{f_c}$	f_c	Ultimate strength of Concrete (from Fig. 7)	$\frac{f_s \text{ (Col. 30)}}{f_s \text{ (Col. 27)}}$	Ultimate Naked Steel.	Percentage.	Yield point Naked Steel.	Percentage.
1 .00	54 600	61 300	29 000	85	36	805	3 100					
2 .00	88 000	99 000	47 000	78	36	1 305						
3 .00		48 500	24 250	70	37	655						
4 .00	92 400	104 000	48 000	98	35	1 370						
5 .00	68 200	77 000	36 600	73	36	1 020						
6 .00	83 000	93 000	43 000	77	36	1 190						
7 .00	74 800	112 000	51 000	78	36	1 410						
8 .00	51 400	77 500	35 700	86		990		$\frac{81}{100}$				
9 .00	56 400	85 000	39 000	67		1 080						
10 .00	58 400	132 000	61 000	88		1 700						
11 .00	57 200	129 000	59 000	86		1 640						
12 .00	57 200	129 000	59 000	86		1 640						
13 .00			21 050	88	22¼	940	3 100					
14 .00			27 760	103		1 230						
15 .00			34 400	127		1 530		$\frac{117}{100}$				
16 .00			29 500	80	22	1 340						
17 .00			37 100	94		1 680						
18 .00			42 100	107		1 910						
19 .00		105 500	48 500	99	21	2 310	2 400		85	57	48	100
20 .00		105 500	48 500	99		2 310			85	57	48	100
21 .00		102 000	47 000	94		2 240			85	55	48	100
22 .00		102 000	47 000	94		2 240			85	55	48	100
23 .00		95 500	44 500	103		2 120			61	73	41	108
24 .00		88 500	41 300	96		1 970			61	68	41	100
25 .00		145 500	44 700	107		2 130			85	53	48	93
26 .00		159 000	48 800	117		2 320		$\frac{100}{100}$	85	58	48	100
27 .00		145 500	44 700	107		2 130			85	53	48	93
28 .00		135 500	41 500	99		1 970			55	49	48	86
29 .00		109 000	50 000	102		2 380			85	59	48	104
30 .00		105 500	48 500	99		2 310			85	57	48	100
31 .00		72 000	33 200	92		1 580			85	39	48	69
32 .00		72 000	33 200	92		1 580			85	39	48	69
33 .00			54 000	59	16	3 360		$\frac{70}{100}$				
34 .00			39 000	81	9	4 350						
35 .00	2 228	3 300	65 500	60	33	1 990	2 700					
36	2 237											
37	2 091							$\frac{65}{100}$				
38 .00	4 985	14 400	61 700	70	28	2 200	2 700					
39	5 095											
40	4 985											

(1)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)
Number.	Maximum Load.	M_B Foot-pounds.	$f_s=\frac{M_B}{A_s d}$	Percentage of Standard B-F-R.	$\frac{f_s}{f_c}$	f_c	Ultimate strength of Concrete (from Fig. 7)	$\frac{f_s(\text{Col. } 30)}{f_s(\text{Col. } 27)}$	Ultimate Naked Steel.	Percentage.	Yield point Naked Steel.	Percentage.
41		7 850	71 000		25½	2 780	2 700		100	71	60	118
42		17 700	72 500			2 840	2 700			72		122
43		33 600	76 000	72		2 980	2 700			76		127
44		77 500	77 500	67		3 040	2 700			77		130
45		26 000	68 500	64		2 680	2 700			68		114
46		50 600	59 750			2 350	2 700			59		100
47		53 200	62 500	68		2 460	2 700			63		105
48		55 500	65 500	71		2 570	3 100			65		109
49		51 400	60 600	73		2 380	2 400	$\frac{72}{100}$		61		100
50		57 600	68 000	82		2 670	2 400			68		113
51		99 000	66 000	70		2 590	2 700			66		110
52		96 200	64 000	68		2 510	2 700			64		107
53		95 200	63 700	68		2 480	2 700			64		106
54		83 500	56 700	65		2 330	2 700			57		94
55		90 800	60 600	58		2 380	2 700			61		100
56		63 000	64 250	77		2 520	2 700			64		107
57		11 450	51 600	57		2 030	2 700			52		86
58		95 650	58 000	120		2 270	2 700			58		97
59	10 500		35 700	77	18	1 980	2 900		60	60	40	89
61	11 100		51 600	82	22	2 350			102	51	66	79
65	10 150		34 500	58	18	1 920			86	40	58	60
66	10 000		51 000	80	23½	2 170		$\frac{81}{100}$	51	100	32	160
69												
71	7 650		41 300	82	18	2 290			60	69	40	103
72	8 650		64 000	97	22	2 910			102	63	66	97
74	8 100		43 800	86	18	2 430			86	51	58	76
75	6 850		56 200	82	23½	2 390			51	110	32	176
76												
77												
78												
79												
81												
82	35 000	67 000	34 600	79	15½	2 230	3 000	$\frac{74}{100}$				
83	22 000	42 000	20 700	68	20	1 040						
86		87 500	38 300	55	26	1 470	3 000				54	71
89		138 000	61 200	68	26	2 360					54	113
92		172 000	69 400	77	26	2 670					54	128
95		112 000	39 600	85	27	1 470					35	113
98		190 000	64 900	70	26½	2 450		$\frac{68}{100}$			54	120
101		130 000	45 200	68	26¼	1 710					54	84
104		102 000	39 400	66	26	1 520					35	113
107		125 000	48 100	57	26	1 850					35	138
110		142 000	40 750	75	26	1 570					54	75
113		221 000	63 300	89	26	2 430					54	117

a span of 48 in.—roughly equivalent to one in a thousand. The curve for No. 16 is parallel with the other curves up to the maximum load, which is practically 20 000 lb. for all three. These curves, it will be observed, are plotted from different origins. After the maximum load Mr. Scott.

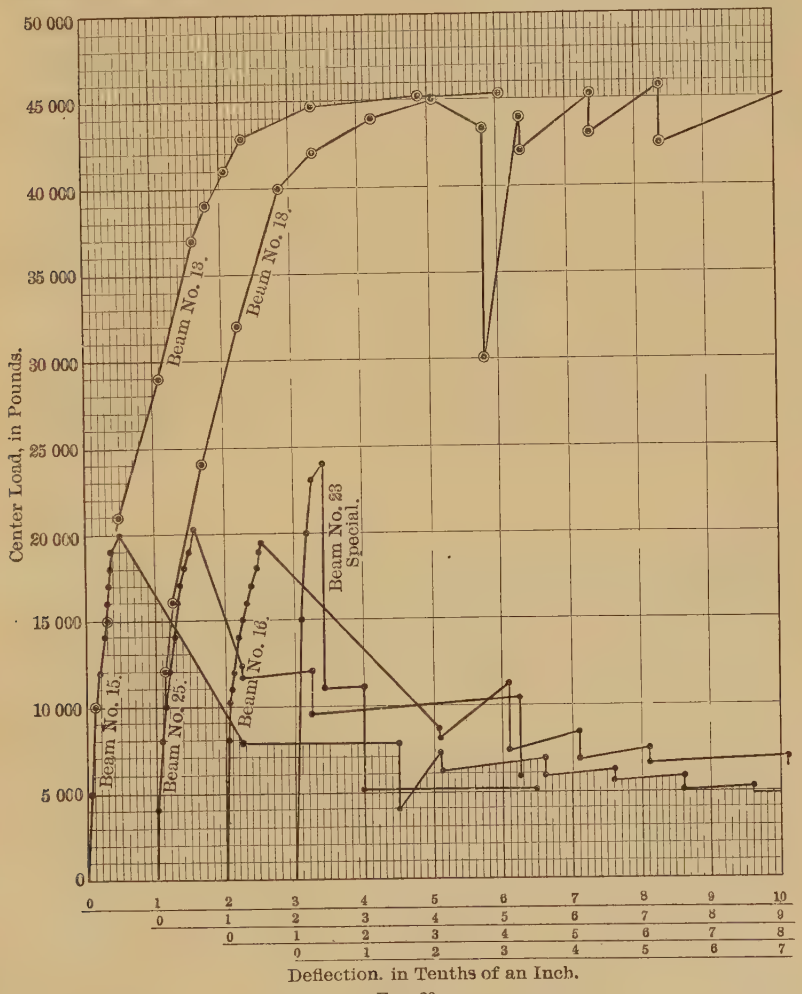


FIG. 29.

was passed, the crack opened more and more, and the supports began to push slowly and gradually away from one another. The drop in the load immediately after the maximum was attained was very much the same for all these beams, decreasing to about half its value while the deflection increased about fivefold. The second stage,

Mr. Scott. shown in Fig. 2, Plate XXI, and Fig. 2, Plate XXII, was in every case after the ¼-in. deflection had been passed. During this stage the steel with the low elastic limit accommodated itself readily to the deflection of the two blocks into which the beam was broken, while the steel with the high elastic limit, not having passed into the plastic state, was stiff enough to form a bow, with pressure on the bottom layer of concrete tending to push it away from the body of the beam, as shown in Fig. 2, Plate XXII. It should be stated that the beams just described were reinforced according to the standard type referred to in the paper. This is a case, then, of equal values for bond failure in beams with rods having the greatest possible difference in yield point. The significance of this fact cannot be mistaken.

When the writer found that there was no doubt about the question of slipping of rods, the question of anchoring their ends engaged his attention. He found the following in Bulletin No. 148 of the University of Wisconsin:

"It has been claimed that beams with straight rods anchored at the ends are stronger than without the anchorage and that the horizontal cracks often observed in the plane of the reinforcement are due to pulling out of the rods. With a view to investigating this, four beams were made with horizontal rods anchored at the ends. The anchorage consisted of a rectangular bar of steel with holes bored through to allow it to be slipped over the ends of the reinforcing rods. The rods were then bent back over this bar so that they had a firm bearing. The loads were applied at 12, 16, 20 and 24 inches from the supports. Plate M-IV [Fig. 1, Plate XXIII] is a photograph of the broken beams. From the picture it is seen that all four beams failed in the manner typical of beams with only horizontal reinforcement.

"Plate M-V [Fig. 2, Plate XXIII] shows the condition of the ends of two of the beams, from which it is seen that as the anchorage was perfectly solid, the rods did not slip at the ends; therefore this crack cannot be due to pulling out of the rods. A comparison of the strengths of these beams with those of Series B is given in Table M-2 [Table 9]. Since the two sets of beams were exactly alike except for the anchorage, the results are directly comparable.

"TABLE M–2. [TABLE 9.]

"*Comparison of strength of anchored and unanchored-rod beams.*

"No. of beam.	"Max. Load.	
	"Anchored.	"Not anchored.
1	3 300	3 500
2	3 600	3 700
3	3 600	4 300
4	3 600	4 900

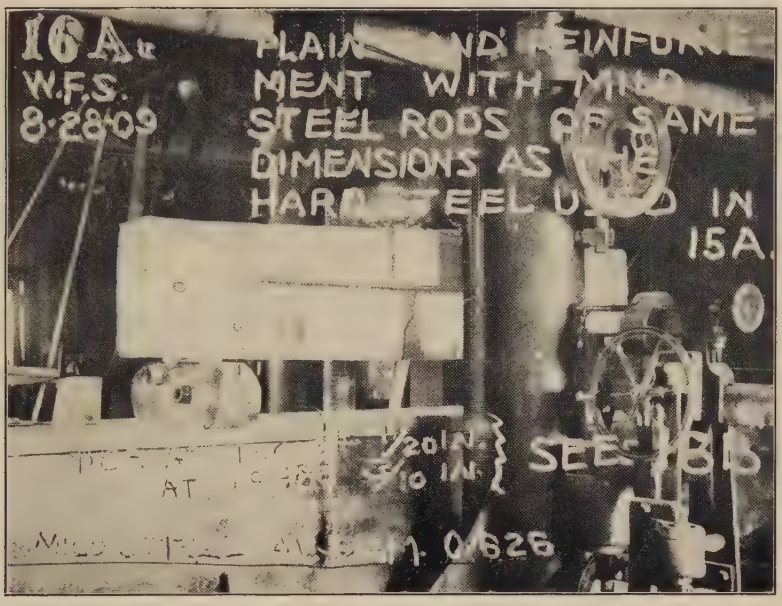

16A
W.F.S.
8-28-09
STEEL RODS OF SAME
DIMENSIONS AS
IN
15A.
SEE

FIG. 1.

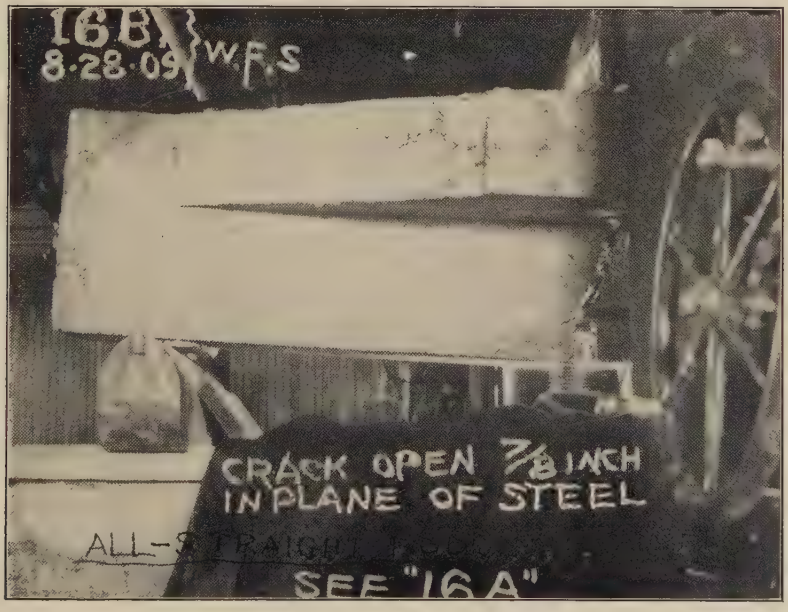

16B
8-28-09
W.F.S
CRACK OPEN 7/8 INCH
IN PLANE OF STEEL
SEE "16A"

FIG. 2.

"It would be rash to draw final conclusions from these few tests, but they seem to show first, that the horizontal crack in the plane of the reinforcement is not due to pulling out of the rods but that it is due to the pulling of the concrete upwards away from the rods, and therefore cannot be prevented by anchoring the rods at the ends; and second, that anchoring the rods does not make the beams stronger with the proportions here used." Mr. Scott.

The writer believes that there is only one cause to which may be attributed the fact that there was no apparent value obtained by anchoring the rods at the ends in this series. This cause is the most important thing to be considered in the study of reinforced concrete; the slipping of the rods was not arrested in its incipiency, and other influences developed almost immediately after the slipping started. The mere hooking of the rods against the back of the plate left sufficient space to permit bond failure to go beyond the incipient stage. Edwin Thacher, M. Am. Soc. C. E., in discussing the paper "Some Mooted Questions in Reinforced Concrete Design," by Edward Godfrey, M. Am. Soc. C. E.,* says:

"By numerous experiments, Mr. Thaddeus Hyatt found that nuts and washers at the ends of reinforcing bars were worse than useless, and added nothing to the strength of the beams."

Here was recorded evidence seemingly establishing the fact that no value could result from anchoring rods. The evidence, however, seemed illogical to the writer, and he was far from being convinced. Therefore, for the purpose of testing the matter experimentally, he reinforced quite a number of beams (see Figs. 29 and 30, and Plates XXIV to XXVIII, inclusive) with various devices which have since been patented by him. The object sought was to develop the full strength of the steel reinforcement. Some of the beams thus anchored failed as quickly as had those without devices for anchorage. Some failed with an increase of strength greater than that for the standard reinforcement, and some of them actually developed the full strength of the steel, as will be seen from Table 8, Plates XVIII, XIX, and XX. Beams Nos. 13 and 18 of this series were reinforced with bars from the same stock as were Beams Nos. 25 and 15, the only difference being that the ends were anchored. The deflection curves for these beams (Fig. 29) were steady up to twice the maximum load reached by the beams without anchorage. Even then, the deflection did not exceed $\frac{1}{2}$ in. in a span of 48 in., or one part in 250, at which time the stress in the steel was more than 80 000 lb. per sq. in., say, 85% of its ultimate strength. The rods were of the same length as those for the standard type, and the beams were poured in the same forms. The photographs show these soon after the maximum load was passed and also at destruction. For

* *Transactions*, Am. Soc. C. E., Vol. LXX, p. 86.

Mr. Scott.

Beam No. 18, the photograph (Fig. 1, Plate XXV) was taken at a deflection of 0.4 in. The beam was left in the machine over night, with the load on, and the next day it was found to have dropped from 43 000 to 30 000 lb. When the machine was started the load went

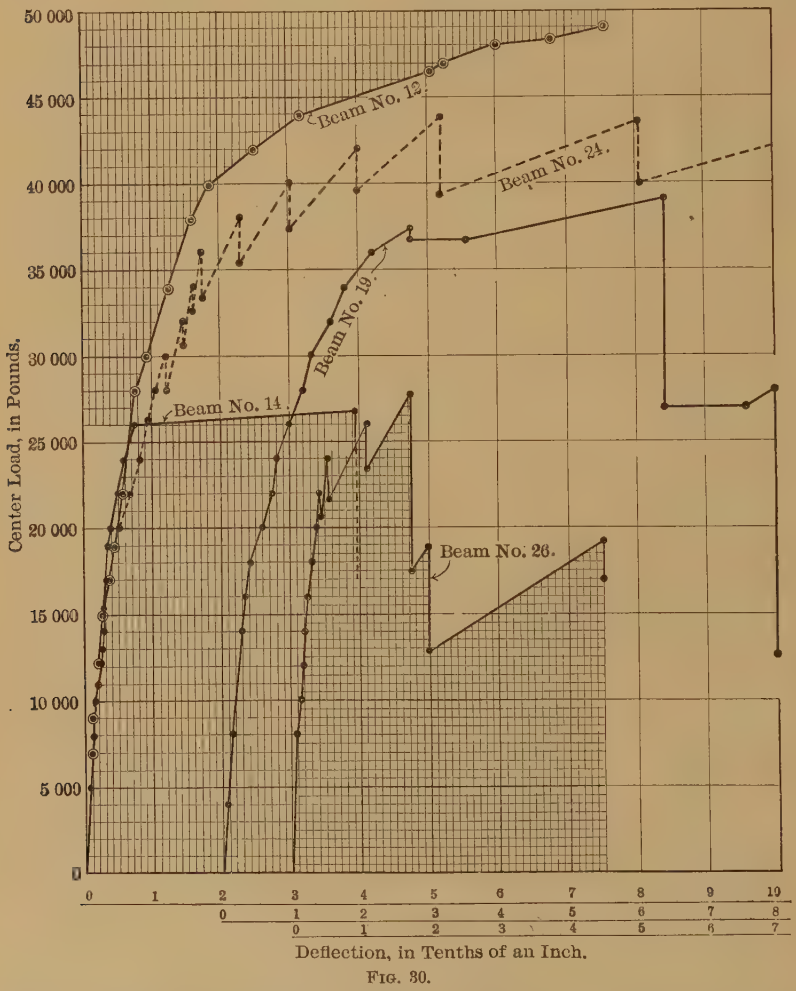


FIG. 30.

back to 43 000 lb., with but slight increase in deflection, and it fluctuated between 42 000 and 46 000 lb. until the deflection went considerably beyond 1 in. Fig. 2, Plate XXV, shows the condition of the beam before it was destroyed by a sledge hammer. The photographs for No. 13 are similar to those for No. 18, with the exception that

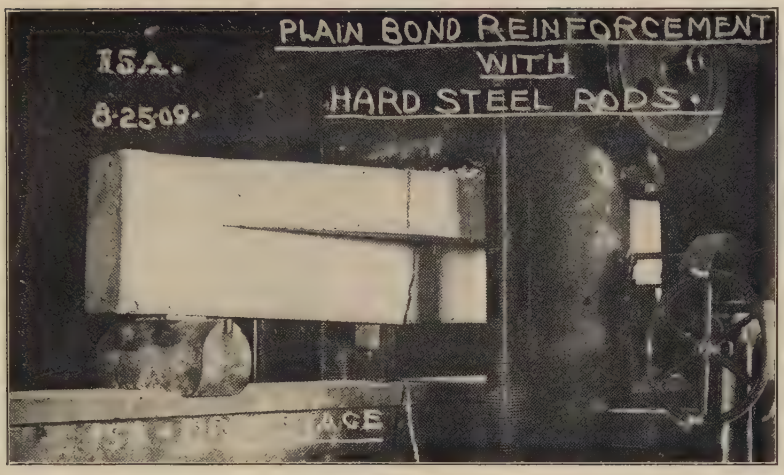


FIG. 1.

FIG. 2.

FIG. 3.

Mr. Scott.

both show an interesting feature which indicates clearly the arching tendency of the concrete. The last photograph (Fig. 2, Plate XXVIII) shows that the lower portion broke away after leaving an almost complete arch of concrete perfectly intact. It is not necessary to describe the other beams, as the photographs show them clearly. It will be noted that the arch of concrete remained intact in every case where the rods were properly anchored.

That the paper gives specific conclusions for practical designing may be seen from a reference to page 253, which says: "For daily office use in reinforced concrete design, * * * Fig. 13 has been prepared." On page 231 the following formula is given:

$$M_b = M_s = f_s A_s d' = f_b \Sigma O l' d'.$$

By transposition, the following standard formula may be obtained for the ready analysis of tests or for the detailed design of reinforcement for beams within the limits of the values given in Fig. 13:

$$f_s^{B\text{-}F\text{-}R} = f_b \left(\frac{\Sigma O}{A_s} \right) l'.$$

The value of $\frac{\Sigma O}{A}$ is a constant for each size of plain or round rod; and an interesting and simple formula for determining this value for any particular size of rod is:

$$\text{Constant} = \frac{\Sigma O}{A_s} = \frac{64}{n}$$

in which n is the diameter or side, in sixteenths of an inch. Thus, if a rod be 1 in. in diameter, or 1 in. square, the constant is equal to $\frac{64}{16}$, or 4. For a $\frac{1}{2}$-in. rod it is $\frac{64}{8}$, or 8, whether plain square or plain round. For deformed bars the writer adopts the value resulting from the equivalent net square or round cross-section. Table 10 has been constructed from these values, and will illustrate the applicability of this constant either in design or analysis.

TABLE 10.

Kind of Bar	¼-in. Rod.		½-in. Rod.		¾-in. Rod.		1-in. Rod.	
	Net area.	$\frac{\Sigma O}{A_s}$	Net area.	$\frac{\Sigma O}{A_s}$	Net area.	$\frac{\Sigma O}{A_s}$	Net area.	$\frac{\Sigma O}{A_s}$
Plain square.....	0.0625	16	0.250	8	0.5625	5.3	1.0	4
Plain round.....	0.0491	16	0.196	8	0.4418	5.3	0.7854	4
Thacher.....	0.042	17.3	0.160	8.9	0.365	5.8	0.630	4.4
Kahn.....	.....	....	0.250	8	0.5625	5.3	1.0	4
Corrugated, old...	.....	....	0.18	9.5	0.37	6.6	0.7	4.8
" new..	0.06	16	0.25	8.0	0.56	5.3	1.0	4.0

Mr. Scott.

The value for the stress in the steel obtained from this formula will represent the maximum that can be developed on any particular beam by the bond-friction-resistance found experimentally, as in Fig. 13, in which it is given for concrete one month old, rods plain and straight, and a constant distance from the compression side, the cross-section of the concrete being constant from end to end. The following additions should be made to these values: Add 10% per month for 5 months more than the one given for age of concrete; add 50% for value of bending half of the bars; add from 5 to 20% for value of deformations (say 10%); add from 10 to 20% for stirrups. If the materials in the concrete have not been proportioned and mixed carefully, deductions should be made, and will vary according to the actual experience of the individual making it. For cases of concentrated loading, the cross-section at the maximum load should be considered for values of β and cot. α, and, for uniform loading, the cross-section at the first quarter point of the beam from the reaction line.

Table 8, Plates XVIII, XIX, and XX, gives the values resulting from the foregoing formula in comparison with those resulting from the actual maximum bending moments in the test beams of prominent experimenters. The reports of these tests, which have been gathered from two continents, have been incorporated in Table 8 without regard to whether they were considered by the experimenters as illustrating failure by compression of the concrete, diagonal tension in the concrete, or tension in the steel. For the reasons given, the writer cannot see any evidence of primary failure being anything other than bond failure in most of the beams the tests of which are there analyzed. After all, this is a question of definition; but, he believes that his definition of the function of reinforcement is at least consistent with the fundamental idea of reinforced concrete. The function of reinforcement, first and always, is to develop the compressive strength of the concrete, and, if it fails in any way to do this, there is something wrong with the reinforcement, not with the concrete. The beams in Table 8 represent practically every common type of reinforcement, quite a wide range of proportions between steel and concrete, a large variety of spans from short to long, and the various types of loading. Only six beams in this entire table, namely, those tested by Thacher, have actually failed by the breaking of the rods in tension. These beams are interesting when examined for the ratio of breadth of concrete in the plane of reinforcement to the sum of the perimeters of the rods. Comparing Thacher's beams with others, it is seen that the concrete between the rods is greater than in any of the other beams. These beams really represent characteristic tension failures of the steel, and, in the writer's opinion, would have been just as characteristically tension failures had the rods not actually broken. That they actually

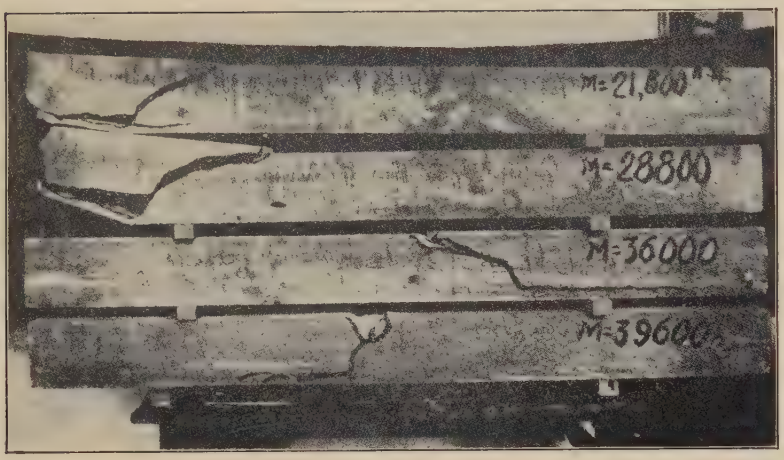


FIG. 1.

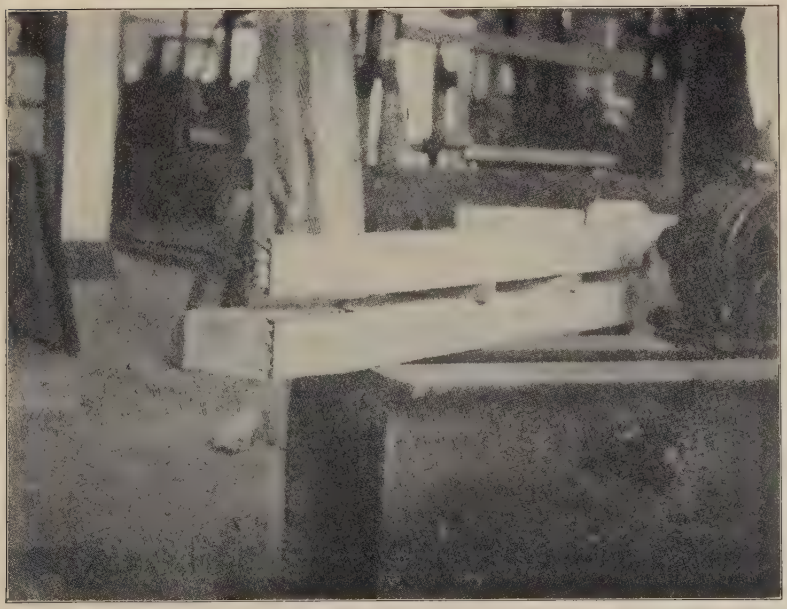


FIG. 2.

did break, is unusual, and may possibly be attributed to a slight brittleness, or some such property, in the material. It should be noted that the steel in most of the other beams given in Table 8 was stressed considerably less than, say, 85% of its ultimate strength, whereas it has been noted that when steel is embedded in a sufficient amount of concrete it may be stressed to more than 85% of the ultimate strength. Thacher reports having stressed it up to the ultimate strength and actually broken the rods. Mr. Scott.

From Column 37 of Table 8 it will be seen that Beams T-5 and T-8 of Professor Marburg's series indicate that the ultimate strength of the steel was very nearly reached. Beams Nos. 132, 138, and a few others, may also be considered as types of actual failure of the steel. It is quite probable that more than 85% of the ultimate strength of the steel was actually developed in the rods in these beams. Considerable judgment must be used in the consideration of Columns 27, 30, 33, 37, and 39. Column 27 is based on the values obtained from Fig. 13, which values were obtained from a most heterogeneous series of tests made mostly by students and reported more or less by men who based their findings on the reports of these students.

Further, very few of these tests were entirely free from foreign influences. The test beams were made under various rules, in the various laboratories in which the experiments were conducted; and the allowances for the influence of deformations, of stirrups, and of bends, more or less approximate the real influence, on account of the lack of rating of the personal equation in these tests. Column 30 is based on the values for a lever arm which was computed on the assumption of perfect bond and that the ratio of the moduli of elasticity is 15. It has been pointed out that this value for the lever arm will increase with two circumstances: the slipping of the rods, and the decrease in the modulus of elasticity of the concrete at its ultimate strength. These are factors which are more or less indeterminate, and are not amenable to definite mathematical treatment. In the latter case the concrete would have to be stressed to nearly its ultimate strength before there would be a material difference from the values given by the ratio of 15.

Under any circumstance, the values in Column 30 bring the question as near to a mathematical basis as possible. The values are purely arbitrary, but they lead to a mathematically correct basis from which to judge of the various modifying influences. For instance, if the indicated compressive stresses in the concrete are much below its average ultimate strength, and if Columns 31 and 35 show a low percentage, it is fair to assume that the beams in the series are below the general standard of fabrication, and *vice versa* if they show a high percentage. When the ultimate strength of either concrete or steel is nearly reached, the percentage will also be low. The values for

Mr. Scott.

the naked steel in Columns 36 and 38 are taken, where possible, from the experiments conducted for each series. Column 39 serves to show how little relation exists between the yield point of the steel and the amount of tension usually developed in beams.

In the paper the assertion was made that hooks at the ends, like deformations, tend rather to decrease than to increase the bond-friction-resistance. Beams I to XII of Table 8 are interesting in that they confirm the writer's contention in this respect. All have about 81% of the standard bond-friction-resistance, while the deformed bar gives even a lower value than those with hooks, or about 70% of the standard.

It will be interesting to observe that American practice differs conspicuously from European with respect to hooks. The small, short hook so common in America is scarcely known in Europe, where custom now favors the hook with a large bend. Needless to say, the latter is much less likely to cause destruction of the concrete than is the former. What is true of the large hook applies also to the large sweeping bend.

Beams I and II of Table 8 illustrate the value that results from increasing the breadth of concrete in the plane of the reinforcement. The only difference between these two beams is that the stem of Beam II was twice as wide as that of Beam I. The reinforcement and concrete are otherwise exactly alike in each beam, yet Beam II carried 60% more load, for no other apparent reason than that the rods had a greater breadth of concrete between them. The writer's bond-friction-resistance values for these two conditions agreed with the experimental determinations to within 8 per cent. Like specimens of fairly uniform material seldom give as close results as this on ordinary tests.

With further reference to these two beams, it might be well to call attention to what Professor Mörsch has to say on this point:

"In technical literature, the idea has been advanced in the effort to obtain much lower unit adhesive stresses, that these diagonal cracks producing failure are due to an overcoming of the adhesion between the steel and the concrete. It is believed, however, that these first three experiments disclose the weakness of that idea."

He then gives as a reason, what the writer believes to be a very strong argument in favor of the theory of bond-friction-resistance, which, of course, has been advanced since Professor Mörsch's book was written. He further says:

"If exceeding the adhesive strength really was the cause of the cracks, approximately equal values of τ_1 should be obtained at the load at which the cracks first appeared in Beams I and II. They differ considerably, however, since the two values are * * * 123 and 235 lb. per sq. in."

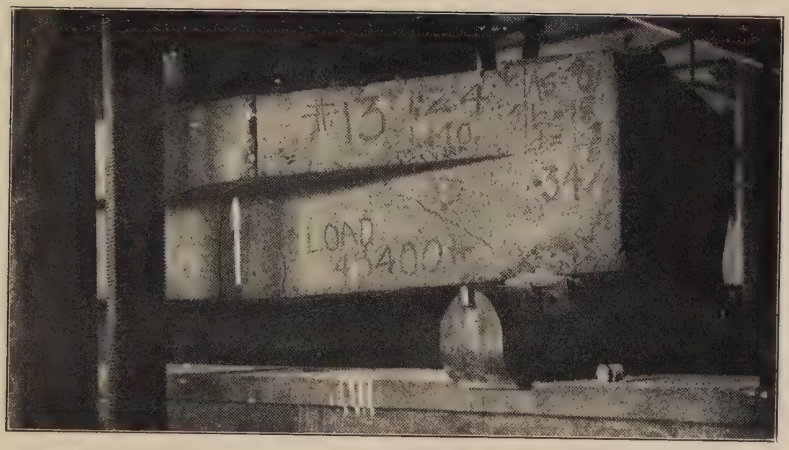


FIG. 1.

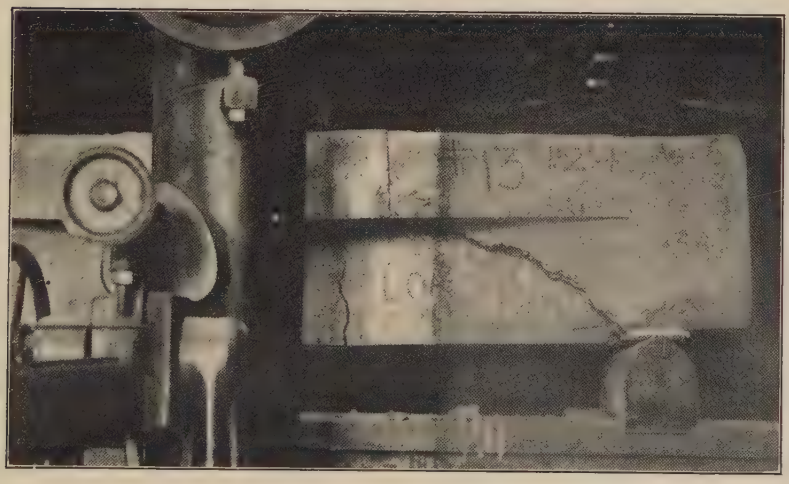


FIG. 2.

He says again: Mr. Scott.

"It is thus seen that the diagonal cracks, which may lead to failure, start at loads which are proportional to the breadths of the beams, and the conclusion is justified that the tensile strength of the ribs in a diagonal direction was exceeded, and that in this case shearing stresses were primarily involved."

Thus he shows that the breadth of the concrete in the plane of the reinforcement is an important factor in the strength of reinforced concrete beams, and although he is inclined to the view that the shearing stresses were principally involved, such a view differs so slightly from that of the writer that the difference becomes really one of terms of definition rather than a fundamental difference of opinion.

After having reviewed the general aspect of this subject, the writer will consider in detail some of the points in the discussion bearing on the question of bond-friction-resistance.

Professor Talbot says:

"But the author does not present any evidence that the bars slipped * * * Without such evidence the usual methods of analysis would be followed."

From these statements it is clear that it is only necessary for Professor Talbot to see that the rods have slipped for him to become a convert to the theory of bond-friction-resistance, because, with such evidence, he will have to develop a method of analysis which would take into account the slipping of the rods. Therefore it is not necessary to review his arguments on this point, as their value depends entirely on the assumption of a perfect bond. As to the "actual elongation," he says:

"But the elongation of the steel itself was not measured, and there is every reason to believe that this elongation, at least through the middle portion of the beam, does not differ greatly from the actual elongation of the concrete alongside of it, and, therefore, that the author's line is greatly in error."

As a preliminary to meeting this objection, it should be said that a further explanation as to both curves, "actual" and "apparent," would help to a better appreciation of their value. The actual and apparent elongation given in Figs. 1 and 2 represents more or less comparative values in a practical way. The values of these curves are very much like other values used in engineering. It is known that they are not absolutely representative over the entire length; but, with a proper use of the imagination, one can see that the curves, from the origin to the yield point, for instance, represent not the elongation of the steel alone, but the elongation resulting from the combination of the steel and the concrete before its bond is disturbed, and possibly before the fibers are broken in tension. In other words, the curve, for

Mr. Scott.

practical purposes, is arbitrarily given the equivalent to the combined value, and, for the present purposes, it serves a valuable end. At some future time some ingenious experimenter may give us the relative work done by the concrete and the steel below this point. Further, the top end of the curve for actual elongation undoubtedly takes a turn at the yield point of the steel much like that for naked steel; but here, again, one must be satisfied with the picture supplied by the imagination until future experimenters get to work on the problem. For the present, there is no occasion for quibbling over these elements of the curve, the positive and certain features being sufficient and valuable. The same is true for the curves for apparent elongation. These curves represent values taken from extensometer readings, or in some cases from direct readings over a small section of the beams between the two load points of third-point loading. The beams illustrated indicate the slipping that was evident over the portion measured; but, as a matter of fact, it is quite certain that the indications would have been more marked outside of the range of the measured portion, for the reason given in explanation of Fig. 25. When Professor Talbot says that "this elongation, at least through the middle portion of the beam, does not differ greatly from the actual elongation of the concrete alongside of it," he contradicts statements coming from the University of Wisconsin, and made under the authority of Professor Turneaure. These statements have been quoted rather fully herein in the consideration of cracks in reinforced concrete beams, and the deductions drawn will serve to answer Professor Talbot and show him that instead of "the author's line" being "greatly in error," his own reasoning is in error because of his belief in a myth.

Again he says:

"Whatever relation there may be between bond resistance and normal pressure, the fanciful conception of a normal component to the bond stress given in Fig. 3 is not borne out by any tests which the writer has seen."

The fact has already been mentioned that Professor Talbot's conception of bond resistance is based on a fallacy, and that, in order to justify his theory, he is forced to compare two unlike things. This being the case, he naturally believes that the "conception of a normal component to the bond stress" was not borne out by any tests which he had seen. His mistake was in basing his deductions on set microscopic observations by students, rather than on combined microscopic and telescopic observations by himself. In the light, therefore, of the evidence herewith presented, it is hoped that he will now see the fact that the normal component in his beam tests was partly in tension, and that to this fact may be attributed the destruction of the foundation of his theories. Professor Talbot further says:

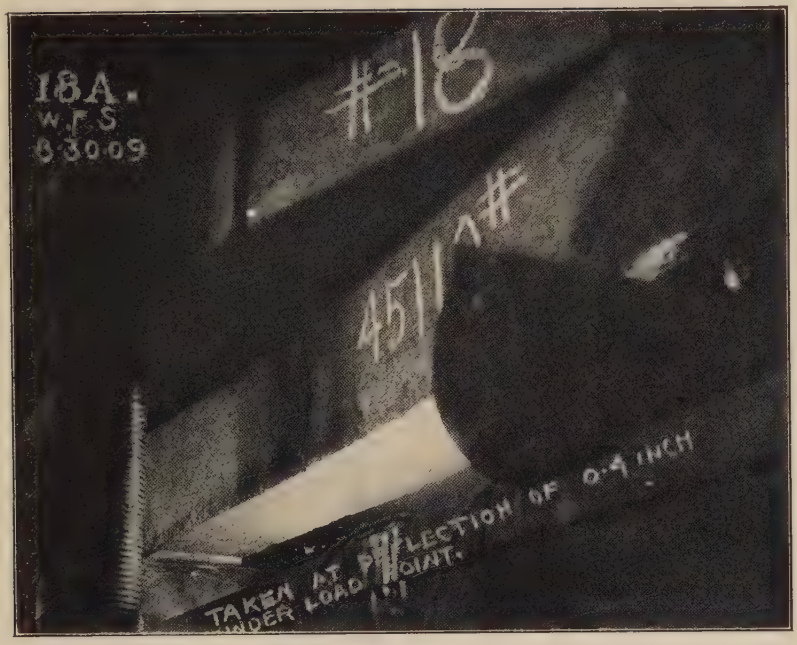

18A.
W.F.S
8-30-09
#18
4511#
TAKEN AT DEFLECTION OF 0.4 INCH
UNDER LOAD POINT.

FIG. 1.

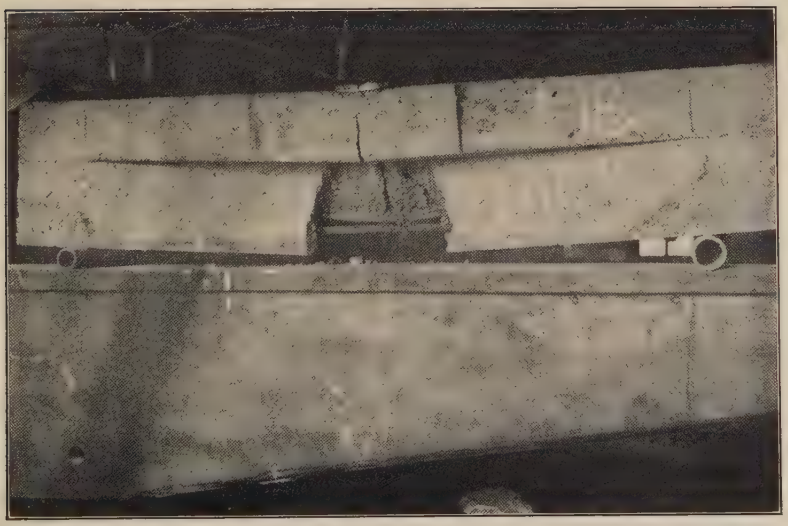


FIG. 2.

"The greater bending moment developed with a single concentrated load than with distributed loads is not explained by the author's statements; it is easily explained on other grounds." Mr. Scott.

In the preparation of the paper the writer believed that Professor Talbot's views on the "Effect of Method of Loading" had a place in conjunction with what he had to say on this question, but he did not include them because of a possible change of heart which Professor Talbot might have experienced in the interval, or that he might experience in the light of the facts therewith presented. However, on the assumption that they are the "other grounds" referred to, they will now be considered. His views* are as follows:

"*Effect of Method of Loading.*—Considerable interest has been manifested by engineers in the effect of method of loading test beams. The opinion has been advanced that a uniformly distributed load will allow a higher moment of resistance to be developed, and cases of tests with loads formed of sacks of sand or bars of iron have been cited in support of this. On the other hand the moment of resistance developed in beams loaded at the middle has been found to be higher than was to be expected if the distribution of stress is as assumed in the ordinary theory of flexure. A few beams were tested with the view of finding out something concerning the effect of changing the point of application of loads.

"These beams had 1% reinforcement (Tables 11 and 12). No. 5 and No. 11 were loaded at the one-third points. No. 21 and No. 30 were loaded at the middle of the span (concentrated load). No. 47 was loaded at eight points, 1½ feet apart, the load being divided equally among these eight points. In Nos. 14, 15, and 18, the load was applied equally at two points 7½ feet apart. The behavior of the beams which were loaded at the one-third points was of course similar to what has already been described. In the beams loaded at the middle, cracks appeared under the load on the tension side early in the test and extended vertically higher until within 3 or 4 inches of the top of the beam at the maximum load. As was to be expected these beams gave steel-tension failures. No. 47, loaded at eight points, failed at the middle by steel-tension, the crack at the middle of the span finally extending vertically to within two inches of the top of the beam. Fig. 32 gives its load-deformation diagram. The beams which were loaded at two points 7½ feet apart failed by diagonal tension in the concrete at the highest loads carried by any of the standard size of beams tested and hence developed the highest vertical and horizontal shearing unit-stress. In general, then, it may be said that all of these beams failed in the manner which would have been predicted.

"The moment of resistance developed in the beams which were loaded at the middle, as calculated from the maximum loads by the usual formulas, was higher than that developed by the other methods of loading, the excess in No. 21 being particularly marked. The fact that in the tests made afterward on steel taken from the beams, one rod of No. 30 was found to have a yield point of 34 600 lb. per sq. in., while those from No. 21 were above 42 000 lb. per sq. in. may be suf-

* Bulletin No. 4, University of Illinois Engineering Experiment Station.

Mr. Scott.

ficient explanation for No. 30 not developing so high a moment of resistance as No. 21. An attempt was made to find the distribution of the stresses through the length of the beams by placing an extensometer on an 8-inch gauged length at the middle of the span in addition to the regular extensometer which was used with a gauged length of 42 inches, but the extensometer device which was rigged up proved not to be serviceable for the purpose, although it did give information of value in other ways. However, it is evident from the deformations measured, as well as from the resisting moments developed, that the distribution of the stresses in a center-loaded reinforced concrete beam is not as assumed in the ordinary theory of flexure. The stress in the steel at the middle of the span evidently is less than the amount given by calculations, and at points somewhat away from the middle the stress in the steel is greater than the calculated amount. The fact that the load is applied at the top of the beam affects the distribution. The results of tests made with center loading by various experiments agree in showing that the moment of resistance developed in tests by center loading is considerably in excess of that obtained from calculations by the theory of flexure and that such results may not be relied on for ordinary loading. Tests of beams loaded at or near the one-third points agree in general very closely with the calculations based on the theory of flexure. The one tested with load applied at eight points compares well with the other loading in a general way, although the moment of resistance developed was somewhat lower than for the beams loaded at one-third points. This was due, in part at least, to the fact that all four of the reinforcing bars in Beam No. 47 were of steel with yield point of only 33 000 lb. per sq. in. It seems proper to say, then, that the results of tests made with center loading are not comparable with other loading and that, as center loading is not ordinarily assumed in designing, this method of loading test beams should not be used. It seems proper to add also, that a loading at the one-third points or thereabout is an allowable method of testing and gives results fairly comparable with uniform loading so far as the development of tensile and compressive stresses is concerned. It does not give as high shearing stresses and hence as high diagonal tensile stresses as uniform loading, but this is counteracted by the fact that with uniform loading these stresses remain high only a short distance from the supports and that in this distance the resistance to such stresses is greater than beyond, and also that at a point one-eighth the span length from the supports these stresses decrease to less than those of the beam loaded at the one-third points. The convenience of the latter method of loading makes it, all things considered, the best general form of loading test beams."

It is not deemed necessary, for the purpose of dealing with the deductions made, to reproduce the tables referred to in this quotation, which is otherwise given in full. For example, he says:

"An attempt was made to find the distribution of the stresses through the length of the beams * * * but the extensometer device * * * proved not to be serviceable for the purpose. * * * However, it is evident from the deformations measured, as well as from the

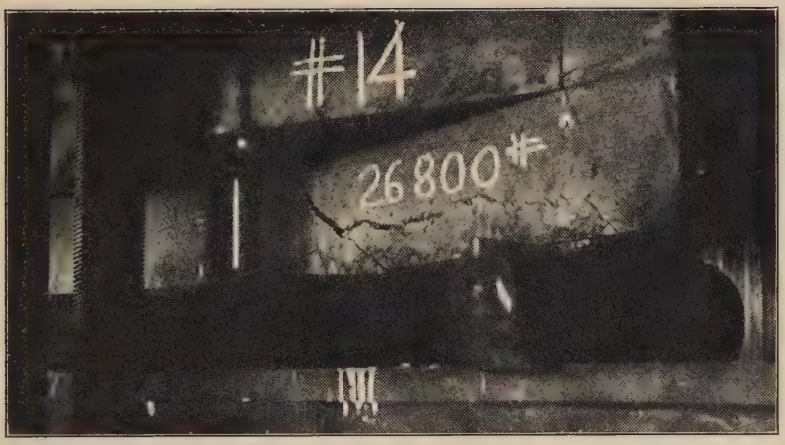


FIG. 1.

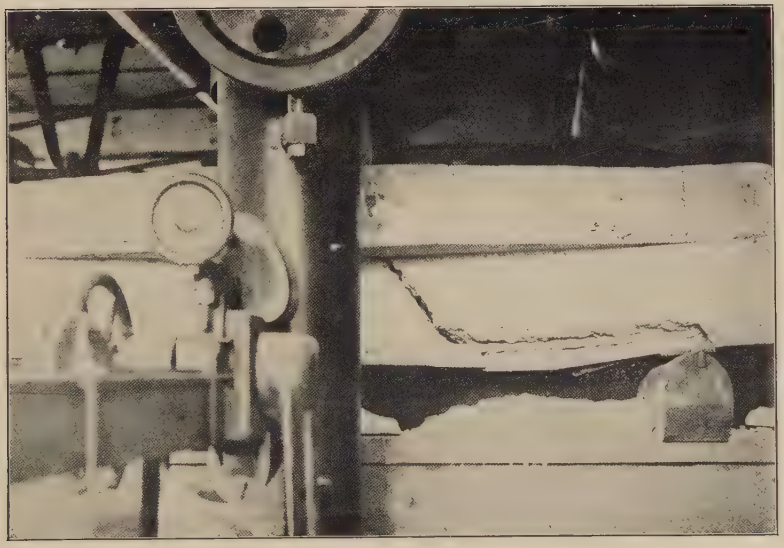


FIG. 2.

Mr. Scott.

resisting moments developed, that the distribution of the stresses in a center-loaded reinforced concrete beam is not as assumed in the ordinary theory of flexure. The stress in the steel at the middle of the span evidently is less than the amount given by calculations, and at points somewhat away from the middle the stress in the steel is greater than the calculated amount."

Here, again, is found superstitious properties attributed to encased steel which are about on a par with those attributed to it by Moritz in the quotation given on pages 300 to 303. These superstitions vanish with the understanding of the fact that the adhesion is imperfect.

By way of a simple example, which clearly represents the comparative effect of a uniform load and a concentrated load in the middle, the test beams of Professor Mörsch are interesting. For instance, Beams IV and V, Table 8, were identical with Beams XI and XII, and the first two were loaded with an average uniform load of 40.15 tons, while the latter two were loaded with an average center concentrated load of 28.6 tons, thus furnishing an actual case where like beams failed in the proportion of 100 to 71, when they were loaded uniformly and when they were loaded with a centrally concentrated load. This uniform load is 1% higher than the value given in the paper. It will be remembered that the writer determined the 70% from a series of Professor Talbot's test beams before this work of Professor Mörsch came to hand.

The value of the angle, α, to be taken for a beam with a uniform load, will depend on the method adopted for the design of such a beam. In the paper the writer left the choice to the reader, it being merely a question of mathematics combined, possibly, with sufficient imagination to see that if, for instance, the weak point of the bond be somewhere in the neighborhood of the quarter point of the span, the stress that will be developed in the steel at the middle point cannot exceed 125% of that developed by the bond-friction-resistance at the quarter point, notwithstanding the fact that the length of rod is 200% of that in the other instance. In this case α must be considered as if a maximum equivalent concentrated load occurred at the quarter point. On the other hand, if α be considered as if the maximum equivalent concentrated load occurred at the middle point, which is the case considered in the paper, the uniform load carried would be 40% greater than the middle concentrated load. The difference resulting from the two methods may be attributed to the variation in the value of the normal component of the bond stress for the two values of cot. α. The writer is quite willing to submit these questions to the decision of the "careful reader."

Professor Turneaure, in opening his discussion, is right in his interpretation of the writer's point of view, and he will find that a "careful inspection of the original sources of information," if

Mr. Scott. gathered by responsible agents, will show that "with very few exceptions, the strength of the beams tested was * * * determined by the bond strength," as primary cause of failure. He will further find that the "other factors, such as tensile strength of steel or strength of concrete," were secondary elements of failure. It will not be necessary to repeat the preceding arguments to show these facts. He further says:

"Referring to these diagrams in detail, Fig. 5 gives the results of certain tests. * * * In the bulletin in which these tests are described it is distinctly stated that three out of four of these beams failed at or near the center as a tension failure; that is, by the overstressing of the steel. The fourth beam failed by diagonal tension or shear. None of these failures was caused by slipping of the rods, and, in the first three cases, where the failures occurred at the center, it was very local in character, and wholly a question of tensile strength of steel and compressive strength of concrete. This is quite evident from the photographs of the beams shown on page 343 of the bulletin."*

Reference to these photographs is made because the writer is firmly convinced that, by viewing them in the light of what he has had to say on the question of how cracks should show themselves if there were no slipping, they will be accepted as strong evidence that slipping was the primary cause of failure. Had these beams been examined at the ends, it would have been found that there was a space between the extremity of the rod and the end of the casing at one or both ends of the beam. The failure of the concrete in compression was secondary failure, and was due to the tipping of the two parts of the beam on their top edges, which caused spalling. He says:

"A still greater injustice has been done in Fig. 6 to the tests made by Professor Howe * * * which * * * constitute an important group."

The analysis for this entire series is given in Table 8, and the reader, therefore, may draw his own deductions from them. It will hardly be necessary to deal with each individual case cited by Professor Turneaure, simply because the explanation for one is the explanation for all. The writer wishes to thank Professor Turneaure for his "regrets that he is unable to agree with Mr. Scott in his analysis and conclusions," and to reply that "from the best study which he has been able to give to the matter" in this discussion, "it is his belief that," with the new point of view presented by him, Professor Turneaure will now be able to see "that the conclusions are" substantiated "by the data." The proof of imperfect adhesion should change his point of view entirely. The writer regrets that he cannot take the time to review each item in the other discussions, but as they are mostly made irrelevant by the assumption of perfect bond, the same

* Bulletin No. 148, University of Wisconsin, 1906.

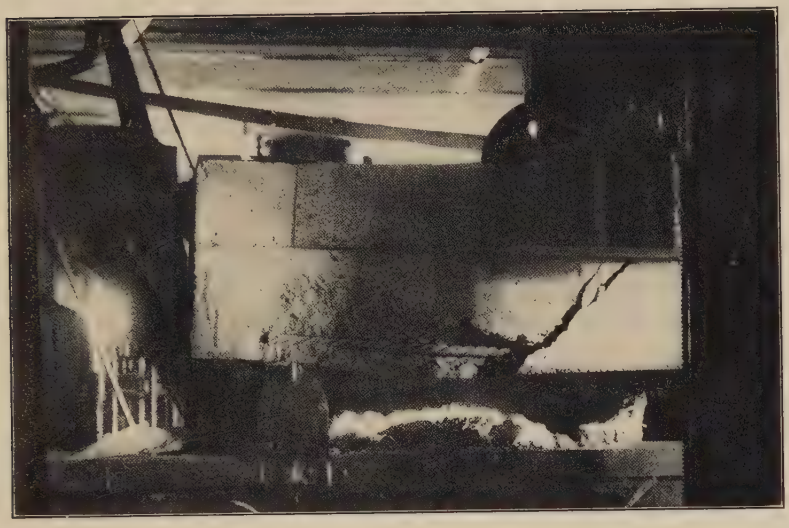


FIG. 1.

FIG. 2.

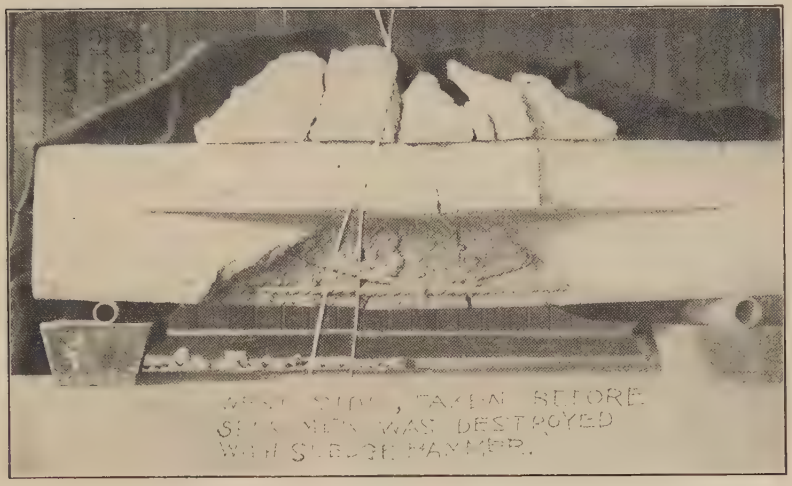


FIG. 1.

FIG. 2.

line of argument would apply with equal effectiveness to them. He wishes to thank Mr. Frye for his discussion, and particularly for the care with which he has reproduced the Darmstadt tests. His elaborate consideration of these tests relieves the writer from making further reference to them, as they speak for themselves. Mr. Scott.

Mr. Godfrey has had a great deal to say about "errors, both original and borrowed," but the careful reader will have seen that the errors he attributes to the writer have absolutely no bearing on anything said by him. In truth, the reverse is the fact. Mr. Godfrey will pardon the writer for giving voice to an impression received while reading his discussion, as it seems to have been written when he was entirely under the spell of his own preconceived ideas, and therefore it does not constitute a normal, unhampered review of what the writer presented for consideration. For instance, what Mr. Godfrey has said in the past or says at the present regarding the question of spacing of rods has no reference to the writer's point of view. He differs only in degree from the theory quoted from Turneaure and Maurer. Professor Turneaure says the rods should be spaced not less than two diameters from center to center, and Mr. Godfrey says four.

As to the question of tension in the concrete, raised by Mr. Godfrey, the answer is the same as that made to Mr. Maguire, and will not require repetition. Mr. Godfrey refers to the tests of L. J. Johnson. The writer had reviewed these tests long before this paper was published, but it never occurred to him that they had any place in this discussion, simply because the load is placed almost directly over the support. The stresses in these might approximate the stresses in the skewback of an arch but they could scarcely be considered as beam stresses.

In closing, the writer wishes to repeat that the Profession is indebted to those eminent investigators who have unselfishly reported their data on experiments. It will have been seen that he is not a blind hero worshipper, and he wishes to add that in taking exception to the views of Professors Talbot and Turneaure, as well as to their many confrères, he appreciates none the less the value of the work they have done. He ventures to hope that the importance of the subject may be held to excuse his having occupied so much valuable space in replying to the discussions, and he cannot conclude without a word of sincere appreciation of the invaluable services in the editing and reproduction of "copy" by the Secretary of this Society and his efficient staff; or further, without acknowledging the services rendered by Mr. Peter Gillespie in aiding in gathering together the writer's notes for this closure, and Mr. K. A. MacKenzie in aiding in making the tests in the Engineering Laboratory of the University of Toronto.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1199

THE GOING VALUE OF WATER-WORKS.*

By Leonard Metcalf and John W. Alvord, Members, Am. Soc. C. E.

With Discussion by Messrs. Clinton S. Burns, Albert I. Frye, Frederic P. Stearns, E. Kuichling, H. F. Dunham, Charles B. Buerger, Halbert P. Gillette, George W. Fuller, Leonard Metcalf and John W. Alvord.

In any engineering project involving ultimate return to the investor in the shape of revenues, there must be considered, not only the expenditures necessary to build the physical plant—with which, of course, the engineer is most directly concerned—but also the investment necessary to put the plant into successful operation and to create the revenues that justify its construction.

This portion of the cost of a project has been termed by the Courts, the "Going Value," or the "Going Concern Value."

Too often the promoters or investors in a legitimate enterprise have either overlooked the importance of providing the necessary funds to cover this cost of the Going Concern, which it will be seen is made up largely of the deficits occurring in the early operation and development of the business of completed plants, or they have under-estimated their amount. In the same way, also, the engineer has failed sometimes to call the attention of the financier to the fact that such deficits will inevitably be encountered in the early days of operation. Such costs not being properly chargeable to physical construction, the engineer, called in by his client to design and supervise the building of the plant, but not concerned with its operation or with the upbuild-

* Presented at the meeting of April 5th, 1911.

ing of its business thereafter, has naturally considered these costs to be outside of his domain or responsibility.

There are circumstances, however, which compel the engineer to give attention to this part of the investment, and which raise the question whether, after all, he is not better fitted to determine these expenses intelligently than is the promoter, the financier, or the owner; for the study of such added values involves primarily knowledge of plant operation, as well as business promotion, a class of information which the consulting and supervising engineer must have, in order to design plants which may be operated economically. The engineer, well versed in such matters, therefore, should be as fair and competent a judge of the going value of a project as of the design and cost of its physical structure.

The necesssity of estimating going value has been forced directly on the attention of the engineer engaged in constructing and operating public utilities by the fact that he is often called on to value the plant and property of a "going" concern. The work of estimating the physical part of the property is not difficult, and usually lies well within the domain of his acquired experience, but when it comes to estimating the value of an already acquired income, involving, as it does, a working knowledge of operating costs, hypothetical growth, estimates of semi-developed revenues, and the like, the engineer finds himself confronted with a problem which demands—in addition to such experience —some knowledge of the law, a judicial attitude of mind, and logical thinking of the highest order.

In the last dozen years or so, the frequent demands for the appraisal and valuation of water-works properties in America, for the purpose of purchase or for rate-making, have compelled the attention of engineers to this difficult subject to a degree perhaps greater in water-works than in any other class of public utilities, so that, while going value is common to almost all forms of human enterprise, and will inevitably be more and more clearly recognized in all cases where complete preliminary estimates or scientific re-valuation are required, yet, up to the present time, water-works valuations have been responsible for most of the effort which has been made to systematize logical investigation of this subject.

Methods for properly estimating going value have been discussed in water-works valuations for more than a dozen years, and many

minds have contributed valuable suggestions which have modified early views and systematized its computation, until now certain general principles, concepts, and courses of procedure for determining going value, have come to be recognized as sound and helpful.

It is not strange, however, that estimators new to the subject, who from time to time find themselves obliged to give it attention, are temporarily involved in some of its complexities, and originate suggestions which have already been well discussed and perhaps shown not to stand the test of reason. This, of course, tends to confuse the discussion. It is desirable, therefore, that what has already been accomplished should be recorded, so that further progress can be made.

In an attempt to record what had already been thought out, as well as to suggest lines for further discussion, there was presented to the American Water-Works Association, in June, 1909, a paper by Mr. Alvord entitled "Notes on Going Value and Methods for Its Computation," to which those interested may refer for a more general and preliminary treatment of the subject. It may be added that a valuable discussion followed the presentation of the paper.

Fully recognizing that lack of specific information and discussions have given rise to honest differences of opinion as to the details of going value determination, the writers offer this paper as a further contribution to the subject, with particular reference to a certain phase of it, namely, the consideration to be attached to the interest account. This item, though of comparative insignificance in the valuation of small plants, is of very substantial importance in its bearing on the going value of large plants.

Definition of Going Value.

Going value may be defined as the value of a created income, or —from the reproduction point of view—the cost of acquiring a given income. Commonly, though erroneously, considered as an intangible value, it is none the less a real element of cost, which must be taken into acccount.

The "going value" of the public service or quasi-municipal corporation is somewhat analogous to the "good-will" of the business corporation. It requires time and investment to build up good-will in business. The operator of the public service corporation knows that, under ordinary circumstances, he cannot hope to meet the operating

expenses and fixed charges of his corporation in the first few years of its existence. It must pass through a formative period during which the losses in operation and fixed charges must be carried until its gross income is sufficient, not only to meet the operating and maintenance expenses, but also to leave a margin sufficient to cover depreciation and fixed charges.

Going concern cost, therefore, is as much a part of the reproduction cost of a public service corporation's property as is the reproduction cost of its physical plant. It appertains, however, to the business, rather than to the physical part of the plant.

As a fundamental principle, it must be clearly understood that cost is not synonymous with value. What a property has cost in the past may be of use in determining what its present value is, but clearly past cost is not necessarily identical with present value; therefore, in determining past cost one should confine oneself to that line of inquiry alone until it is completed, and in reproducing a plant it is only proper and logical to consider similarly what it would cost to rebuild it under present conditions. The two should not be confused. Both may aid in determining value. Neither is value of itself alone.

The Wisconsin Public Service Commission, in one of its recent decisions, has taken as a measure of going value, in a reproduction of the property, the past cost to the company, in operating deficits, advertising, and other expenses incident to building its income up to the point where it is sufficient to carry the depreciation and fixed charges, as well as the operation and maintenance expenses of the project. As applied to the reproduction method, this interpretation of going value is erroneous, because it deals with the "original" cost, and not the "reproduction" cost, of developing the present income of the plant; and because, while admitting that original cost is significant in a determination of the value of the corporation's property, the Courts hold that it is not of controlling importance, and have laid down the precept that the reproduction cost, as of the date of valuation or taking of the plant, shall be accurately determined as one of the standards of present value.

If such is the case with reference to the physical plant, why not with reference to the business of the corporation? Obviously, it should be so, for a moment's reflection will make clear that, with growth in population, the company has to face constant demands for new capital,

to meet the growing needs of the community, and this new capital passes through a period when it does not earn an adequate return, in exactly the same way as occurred in the case of the capital originally put into the plant. The going value of the plant, therefore, under normal conditions, should increase with the lapse of time, and with increasing investment in plant and business.

The Measure of Going Value.—What, then, is the measure of this going value, and what elements influence its amount?

Without going too minutely into a history of the development of the methods for the determination of going value, already fairly well established, suffice it to say that in a general way it has been found necessary to measure this element by a hypothetical comparison. This is done by determining the difference in earning capacity between the existing plant and a hypothetical new plant, which, for convenience, may be called "The Comparative Plant." This comparative plant is assumed to be built, beginning with the date of valuation, or taking, and to acquire business up to the level of the existing company as rapidly as possible. The sum of the present worths of the annual excess in net return of the existing plant over the hypothetical or comparative plant, in the period of years from the date of taking to the time when the earnings of the comparative plant are assumed to become identical with those of the existing plant, represents the going value of the existing plant.

The sum thus obtained represents the additional amount which a purchaser could afford to pay to the owners of the existing plant for its established income, over and above the cost of its physical property.

This method of arriving at going value may be termed the "Comparative Method."

The Investor at the Parting of the Ways.—In the case of a large plant, in which the period of years required for the reproduction or building of its physical property would be long, it will be helpful, in analyzing its going value, to place oneself in the position of an assumed capitalist, who stands, on the date of valuation, in the position of having in hand the necessary capital, so that he could either buy the existing going plant with its established business, or build a new hypothetical or comparative plant to replace the existing one under non-competitive conditions. In brief, this imaginary capitalist may be considered as standing at the parting of

the ways, with a choice of two paths, and as desiring to know from his engineer their relative financial desirability. This requires the engineer to determine for both cases the probable return which the capitalist would receive on his investment, during the period of years assumed as fairly necessary for a comparative plant to acquire the growing business of the existing plant.

Knowledge of operating conditions and growth of plants under different circumstances allows this comparison to be made with greater certainty than might be supposed. It is necessary, however, to be strictly judicial and thoroughly logical, in order to produce reasoning which will stand the test of cross-examination.

Some of this reasoning will be followed:

Should the capitalist take the first path, and buy the existing plant, he will evidently receive as return on his investment the net income indicated by its past history, modified by such circumstances as will be likely to influence its net income during the period when a new comparative plant can reasonably hope to overtake its development.

If he chooses the second path, that is, to build the new hypothetical, or comparative plant, he must be assumed to have in hand at the date of taking, a capital sum equal to the full reproduction cost of the existing plant and its business. This capital he must keep in such ready, or convertible, form as to enable him to meet the hypothetical costs and expenses as they arise.

Obviously, on this second path, as a builder of a new plant, he will not have to use the full amount of his capital at first, but will make only such expenditures as may conform to the reasonable construction requirements from year to year, until the end of the construction period of his comparative plant, and thereafter, such additional expenditures, up to the end of the going value development period, as may correspond to the cost of developing the business or going value of a newly starting company.

The capitalist then finds himself, in this second path, in the position of still having in hand at the end of the first year, a considerable proportion of his capital, on which portion he will be able to earn a rate of interest commensurate with the need of keeping it in convertible form. For comparative purposes, however, he will lose the return on the money expended during the first year for construction requirements.

Similarly, for the second, third, and additional years of construction and development of going value.

Clearly, if the existing plant is small, so that the construction period of a comparative plant will not exceed one year, the amount of unemployed capital on which interest can be earned, pending its use for construction purposes or acquirement of business, will be small, perhaps negligible, and will have little effect on the going value of the property in question.

In the case of a large plant, however, it becomes an element of importance, substantially reducing the amount of the going value which would obtain, were this return on unemployed capital neglected, and, therefore, it must be given careful consideration.

It remains, then, to compare these two paths in terms of returns on the two plants, through the period required for the development of the comparative plant. To do this, first find the differences in the return, under proper accounting methods, on the existing plant, in the light of its past history, and on the comparative plant, under reasonable assumptions as to development of income, including allowance for depreciation in both cases, but excluding consideration of the interest charges; then, to the sum of the present worths of these differences, add the sum of the present worths of the annual return on the unemployed capital. The result thus found should logically measure the going value.

It is evident that interest should not be included as an operating charge, for the reason that the relative return on the two plants, of which interest is a part, is the thing sought. To include interest or profit on capital (invested in plant, in contradistinction to unemployed capital) as an operating charge, would be to charge to operating expenses a portion of the very return which it is desired to value, instead of crediting it to the capitalist as a part of the return to be derived on the two comparative investments.

It is of importance at the beginning to understand clearly that all accounts with the capital under consideration must be begun at the date of taking, or at the parting of the ways, as it has been called, because the discussion of return is wholly a comparative one from this point of view. Some confusion has resulted, even on the part of those familiar with the subject, due to lack of appreciation of this fact.

The writers, therefore, will proceed to discuss more in detail this

question of return, by which is meant, in the case of the existing plant, the difference between gross annual income and operating, maintenance, and depreciation charges, excluding interest charges, however, and in the case of the comparative plant the same difference plus the income derived from the unemployed capital.

In discussing the development and operation of the comparative plant, the following considerations are of importance:

First.—The rate of interest for readily convertible capital is low. It will vary with the length of the period of construction, and to a lesser degree with the length of the period of acquisition of the business of the existing plant by the new comparative plant. For the bulk of the unemployed capital, held in readily convertible form, but awaiting use for construction purposes, 5% is too high a rate in most cases. Probably 4%, more or less, depending on local conditions, is a fairer rate. For that portion of the unemployed capital which is to be used for construction purposes during the year, unless its amount is very substantial, 2% a year, or thereabouts, on the average balance in bank, would seem to be a fair allowance.

Second.—Bankers will charge a discount, or brokerage, for the service of holding funds in readiness for use as required by construction needs, which will tend to decrease the net rate of return on the unemployed capital.

Third.—While this discount for keeping funds in readiness would reduce the average rate of interest, if the amount on deposit during the year is substantial, a portion of the bank balance can be kept in the form of certificates of deposit, on which a rate of interest higher than 2% should be earned. If the anticipated annual expenditures in plant construction are very large, a careful analysis may be made of the probable bank balances as affecting the rate of interest which might be allowed thereon by bankers. Under these assumptions, the average rate of interest which would be earned by the capitalist on his unemployed capital, less discount, would probably vary between 3½ and 4 per cent.

Fourth.—The larger the rate of interest allowed on this unemployed capital, the smaller the resulting going value of the existing plant. The actual effect (in percentage) on going value, is of lesser importance than would perhaps appear at first thought, for the reason

that the period during which there is unemployed capital on hand is small, as compared with the total going value development period.

In its more general aspect, the comparative method of computing going value may be said to conform to the actual experience of the investor in public service properties, and, to some extent, to the condition which he would have to face were he to build a rival plant, rather than to buy an existing plant. Such is the case when a city, desiring to duplicate a public utility, rather than to purchase the existing property, overlooks or willfully neglects the fundamental economic conditions which it must meet, assuming that a reproduction of the physical property is all that is necessary, without allowance for going value. Under such circumstances the property is reproduced under competitive conditions (contrary to the non-competitive fundamental hypothesis of the "comparative" method), which inevitably builds up the going value to large proportions. This has been the actual experience of certain cities such as Mobile, Ala., Sioux Falls, S. Dak., Newark, Ohio, and others.

The general theory of going value conforms strictly to the reproduction cost theory, and makes no assumption as to what the existing plant should earn on its investment, but is based on the actual past return derived by the existing plant and the probable return in the immediate future.

Furthermore, the results obtained by the application of this comparative method appear to comport with the range of going values, common in enterprises of this sort.

Principal Factors Affecting Going Value.

What, it may be asked, are the factors most affecting going value?

In the comparative method, aside from the relative gross earnings, and operating and maintenance expenses, the factors influencing most the magnitude of the going value are:

1. The period of construction required for building the assumed new hypothetical, or comparative plant;
2. The going value development period, or the period from the date of taking to the time at which it is assumed that the comparative plant may overtake the growing business of the existing plant;

3. The rate of acquisition of income by the comparative plant, during the going value development period; that is, from the date of taking to the time when the comparative plant is assumed to have reached the same stage of development of business as the existing plant;
4. The stage of development of the existing plant; that is, whether it may be said to be in a condition where its income is truly commensurate with the degree of development of its physical plant, instead of being either over-built or under-built;
5. The difference in annual depreciation charges assumed for the two plants;
6. The rate of return on the unemployed capital to be used in building the comparative plant, estimated as reasonable for capital which must be kept in a readily convertible form for application to construction.

Period of Construction.—The period and rate of construction of any comparative plant will be determined by the ability to plan and build wisely and economically. Experience has shown that, except in cases where the work is so scattered as to make possible its subdivision into a large number of contracts, the annual expenditure of a sum in excess of $1 000 000 is not an easy matter. On the other hand, it must be admitted that the desire to earn a return on the investment at the earliest possible moment, constitutes a strong incentive to push the work to completion as rapidly as possible, after its inception.

Order of Construction.—The order of construction will be influenced largely by the desire for the early development of income, and it is even reasonable to assume in some cases that temporary structures may be built, with this end in view, and abandoned after the completion of the permanent plant. The recent experience in building the water-works for the new city of Gary, Ind., furnished a striking example of this, for there some temporary pipe lines, temporary wells, and a temporary pumping plant were built to serve the city during the construction of the permanent water-works. These temporary structures were afterward abandoned, but they had served not only the economical purpose of making more convenient the work of the builders of this magic city, but also of reducing the cost of the going value of its water-works.

Going Value Development Period.—Obviously, the longer the period assumed as necessary for the development by the comparative plant of the growing business of the existing plant, the greater will be the going value of the existing plant. The effect of this factor is less marked than that of the rate of acquisition of income during this period, for the reasons that the differences in income decrease rapidly, and that the going value is not determined by taking the sum of the annual differences in net income, but of their present worths brought back to the date of taking or valuation, and the present worth factors become less and less as the period of years required for the development increases.

The past history of water-works, many of which were built during the Eighties, is not a fair criterion for the period required for the acquisition or development of business by water-works to-day, principally for the reason that the public is now educated to higher standards of living, and to the use of water under pressure, which would result in much more rapid development of income by water-works now than formerly. While some have urged that a water company is entitled to be credited with the cost involved in educating the public up to modern standards, was not this cost rather in the nature of an operating cost, than of a capital expenditure, inasmuch as this advantage immediately accrues, without cost, to any newcomer in the field, and cannot be held as a monopoly or asset by the existing company? It is usually assumed, therefore, that the public is educated to the use of water, and that the cost of that education is lost to the existing plant.

Rate of Acquisition of Income.—The rate of acquisition of business by a comparative plant is of the greatest importance, and especially so is the rate of development assumed for the first two or three years of its operation.

Several theories bearing on the rate of acquisition of the business of the existing plant in the comparative method have been advanced, the most important of which are as follows:

(1) The theory that as soon as a portion of the comparative plant is completed, the portion of the existing plant corresponding to it goes out of use.—Thus when the first stretch of distribution pipe system is laid, after the completion of the water supply system, the corresponding part of the existing company's system is assumed to be out of

service. The abutters on this pipe line have then the option of taking water from the new company, or of making other arrangements for obtaining their own supply.

Obviously, under such a drastic assumption, logical, but purely hypothetical, the rate of acquisition of business would depend substantially on the rate of construction of the plant, and, in the business section of the city at least, the income of the existing company would be derived by the comparative company very rapidly. In residential portions of the city, it is conceivable that the existence of wells and other facilities for obtaining water might result in a slower rate of acquisition of business.

(2) The theory that on the date of valuation, or taking, the existing company is notified that the city will build a new plant to replace the old, and that on the completion of the new system, the franchise and contract rights of the old company will be terminated.—Under such an assumption, the income of the old company would be much more gradually developed by the new company, than under the first theory, as many takers would continue to support the old company, as long as it was in business. Many of the old takers, however, would connect with the new company's mains and give up the old supply.

(3) The theory that the community now being served by the existing plant is without public water supply, and has never enjoyed the service of water-works.—Under such an assumption, substantially no service connections exist, and only those houses have water-works and plumbing fixtures as are assumed to have had their own individual water supplies. In substance, therefore, it is assumed that the comparative or starting plant is built in a new field, though the people of the community are assumed to be educated to the value of public water service under pressure, and may reasonably be expected to make connections with the mains of the comparative company and take water therefrom, as soon as they can prepare their houses for it. While this theory recognizes that the education of the public to the advantages of a public water supply has cost the existing company a substantial amount, it does not recognize that this cost should accrue to the company as an asset which it can hope to retain.

(4) The theory that the limiting rate for the acquisition of business by the comparative company is the speed with which service con-

nections can be made and pumping fixtures introduced.—This theory has little to commend it logically.

It will be observed that under none of these theories is the competitive theory developed; and with reason, for such an assumption would imply conditions entirely foreign to those under which the existing company, the going value of which is under determination, is operating, and would therefore not be a correct gauge or standard for determining its going value. Moreover, whether the danger of competition be eliminated by the character of the franchise, or whether it be prevented by the strength of the position of the existing company, or by the past experience of cities which have built competitive water-works, the water-works business in a large community is in fact virtually a controlled monopoly, and the best interests of the public dictate that this should be so. It might be added that control by an intelligent and honest public service commission is to be preferred, alike from the quasi-municipal corporation's point of view and that of the public, which it serves.

Momentary Stage of Development of Old Plant.—The going value of the existing plant will be somewhat affected by the relative stage of its financial development, or amount of its income, as compared with its momentary physical development. It must, therefore, be noted whether the existing plant is in what might be termed a "normal condition" or ratio of earnings to value of property, or whether it is in effect "over-built" or "under-built"; that is, whether, on the one hand, the company has recently incurred construction expenses in improving its facilities to which its income has not yet grown, or, on the other hand, has failed to make necessary improvements to keep the plant in first-class condition, thoroughly up to modern standards and the reasonable needs of the community which it serves.

Thus, one may recognize three conditions in which a plant may find itself:

(1) That of the "over-built plant," which finds itself in a position unable to earn operating and maintenance expenses, depreciation, and full interest charges and return upon its value, as a result of recent heavy capital or construction expenditures, unless perchance the return earned on the value of the plant prior to these construction expenditures was so unreasonable as to make the rate reasonable when the return is applied to the greater capitalization. In the final

analysis of value, weight should be given to this condition of the over-built plant, as well as to the past financial history of the company and existing conditions which may bear on the ability of the company to grow in income to the new capitalization; but it cannot be recognized in computing the cost of reproducing the income, or going value, of the existing plant.

(2) That of the "normally developed plant," in which the financial return is commensurate with the value of the property and the security or hazard of the business.

(3) That of the "under-built plant," the physical condition of which is obviously below a reasonable standard for the community which it serves, in which it is fair to assume that the existing net income will be substantially reduced by the expenses incident to the additional capital expenditures required to bring the plant to a rational standard of water-works construction or service. Here, again, the past history and present conditions, referred to above, are of importance.

Effect of Difference in Annual Depreciation Charges.—As regards the effect on going value of the relative annual depreciation estimated for the two plants, it is reasonable to assume depreciation to begin, in the case of the new comparative plant, on the completion and beginning of operation of its several component parts, which constitute workable units. In the case of a small plant, requiring not more than one year for construction, the contribution to the depreciation account or fund will not begin until the end of the first year after the completion of the construction of the entire comparative plant. In the case of a large plant—such, for instance, as that in which a period of construction of from 5 to 10 years would be required—it is reasonable to assume that contributions shall be made to a depreciation fund by the several portions of the plant as they are completed and put into operation; thus, assuming that a driven-well system, or pond intake, and a portion of the distribution pipe system had been completed as rapidly as possible, in the interest of developing an early income, depreciation should be charged to these portions of the plant on their completion and use, without waiting for the final completion of the entire plant, which might follow several years later.

The Rate of Return on Unemployed Capital.—The reasonable rate of return on the unemployed capital, which the builder of the comparative plant is assumed to have in hand, has already been discussed.

From 3½ to 4%, more or less, was assumed to constitute a reasonable return, in the eastern part of the United States, on capital invested in sound, readily convertible securities, and 2%, or thereabouts, on full calendar months, on unemployed capital carried on deposit in the bank, subject to draft at all times during the course of the year. The obvious effect of the magnitude of the rate of return on the unemployed capital is to reduce the going value of the existing plant. The higher the possible return on such convertible investment, the lower the going value.

Comparison of Conditions of Operation of Existing and Comparative Plants.

In making comparison between the operations of the existing and the comparative plants there must be considered:

(1) The relative income derived both from public and private sources;

(2) The relative operation and maintenance expenses; and, under this item, taxes and repairs are to be included, as well as the other items generally covered by the operation and maintenance accounts, omitting interest or return on capital invested in plant;

(3) The relative depreciation;

(4) The relative net revenue.

In determining the financial operations of the existing plant during the going value development period of the comparative plant, the past financial history and the conditions under which the plant is to operate must be analyzed carefully.

In determining the corresponding figures for the comparative plant, due weight must be given to the studies made of the operation of the existing plant, as well as of the conditions surrounding the comparative plant. The net reproduction cost estimate of the existing plant (after deducting accrued depreciation) will determine the construction cost to be assumed for the comparative plant.

Limit of Capital Account Charge for Interest During Construction.

In the construction cost of the comparative plant, as of the existing plant, there must be included an allowance for loss of interest during

construction quite independent from the interest "return" which will be earned later on the property of the company.

This apparent duplication of interest has been the source of some confusion in the minds of those who approach this subject for the first time. It will clear up the seeming paradox, however, to remember that interest-during-construction payments are inevitable in all projects; they cannot be avoided; they constitute a forced expenditure, alike with the cost of the bricks and mortar, the pipes and the pumps, and cannot be recovered in any way except by addition to capital account or floating debt. Interest during construction, therefore, is logically not a "return" or income in any such sense as one speaks of return, in comparisons for determining going value, but essentially a construction cost. The confusion has resulted only because the word "interest" has a different significance in the two cases.

The dividing line between interest chargeable to the item "interest loss during construction," which is added to construction cost, and "interest" which is return on the investment of the company, is drawn at the point where the several parts or units, making up the plant as a whole, are completed and put into operation, and thus begin to earn some income or return, instead of being a dead load to the company. On the other hand, it must be borne in mind that the plant cannot be expected to earn the full amount of the fixed charges, in addition to operating, maintenance, and depreciation charges, immediately on beginning the operation of the several parts of the plant, and that the losses during such formative period must therefore be carried to capital account, through going value, in precisely the same way that interest during construction must be carried to construction account. There is no other means of compensating the builder or investor for these early losses.

It would be as unfair to the owners of the public service corporation to fail to permit them to recover these early losses, as to make the public pay, through the agency of the rates, an annual return forever after the completion of the works, on excessive, wasteful, and foolish construction.

It is clear, therefore, that, after operation has begun, the early losses in interest on the investment or value of the plant form a considerable part of the going value, and that loss of interest during construction, up to the point of operation, must be credited to capital account as a construction expense.

Application of Principles Outlined.

Assumptions as to Existing Plant.—For the sake of illustration, and to make clear the principles which have been discussed, the writers have assumed a typical case in order to show the steps necessary to determine the going value of a large property on the comparative method herein outlined.

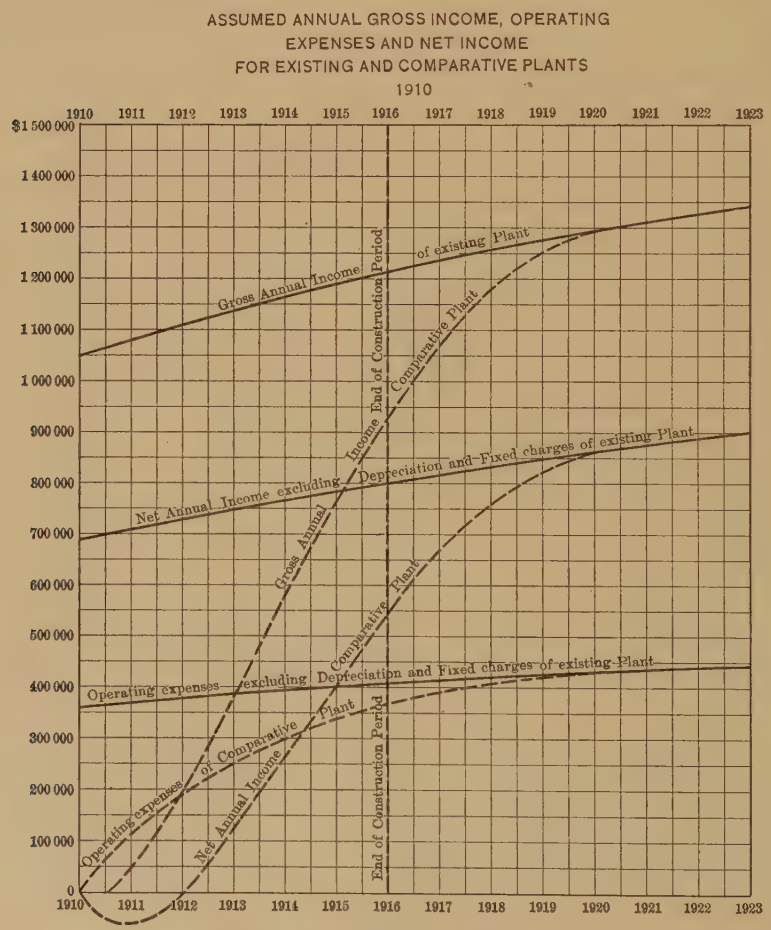

ASSUMED ANNUAL GROSS INCOME, OPERATING
EXPENSES AND NET INCOME
FOR EXISTING AND COMPARATIVE PLANTS
1910
1910 1911 1912 1913 1914 1915 1916 1917 1918 1919 1920 1921 1922 1923
$1 500 000
1 400 000
1 300 000
1 200 000
1 100 000
1 000 000
900 000
800 000
700 000
600 000
500 000
400 000
300 000
200 000
100 000
0
Gross Annual Income of existing Plant
Net Annual Income excluding Depreciation and Fixed charges of existing Plant
Operating expenses excluding Depreciation and Fixed charges of existing Plant
Income End of Construction Period Comparative Plant
Gross Annual Income of Comparative Plant
Net Annual Income of Comparative Plant
Operating expenses of Comparative Plant
End of Construction Period

Fig. 1.

The plant which has been taken for illustration is an imaginary one, supposed to serve a population of 300 000 persons, but care has been taken to make the data consistent and conformable to figures found in operation. This statement is made, however, that the reader

may not be misled into supposing that the figures have been copied directly from the report of any public or private water-works.

In Fig. 1 is shown a forecast of the assumed probable gross annual income, net annual income (excluding depreciation and fixed charges), and operating expenses (excluding depreciation and fixed charges), of the assumed plant, called "the existing plant," which is supposed to be based on the past financial history of such plant, and the assumptions contained in Table 1.

TABLE 1.—General Statistics Relating to Assumed Water-Works Plant.

"The Existing Plant."

Year ending December 31st.	1910.	1915.	1920.
Population	300 000	330 000	365 000
Miles of pipe, all kinds	480	520	570
Population per mile of pipe	625	635	640
Taps in service (live)	40 000	45 000	51 000
Per 1 000 population	133	136	140
Per mile of pipe	83	86	88
Persons per tap	7.5	7.3	7.16
Consumption, annual, in millions of gallons	10 950	13 250	15 980
Daily, in millions of gallons	30	36.3	43.8
Per capita, in gallons per day	100	110	120
Per mile of pipe, in gallons per day	62 500	69 800	76 900
Per tap, in gallons per day	750	807	859
Hydrants (number)	3 840	4 420	5 130
Per 1 000 population	12.8	13.4	14
Per mile of pipe	8	8.5	9
Gross income (total)	$1 050 000	$1 190 000	$1 296 000
Per capita	3.50	3.61	3.55
Per mile of pipe	2 190	2 290	2 270
Per tap	26.25	26.40	25.40
Per million gallons consumption	95.90	89.90	81.10
Gross income, exclusive of hydrant rentals	$896 400	$1 013 200	$1 192 320
Per capita	2.99	3.07	3.26
Per mile of pipe	1 870	1 950	2 090
Per tap	22.40	22.50	23.40
Per million gallons	81.90	76.50	74.60
Cost of operation, exclusive of fixed charges and depreciation	$360 000	$402 000	$430 000
Per capita	1.20	1.22	1.18
Per mile of pipe	750	773	754
Per tap	9.00	8.94	8.44
Per million gallons	32.90	30.30	26.90
Percentage of gross income	34.3%	33.8%	33.2%
Net income, exclusive of fixed charges and depreciation	$690 000	$788 000	$866 000
Per capita	2.30	2.39	2.37
Per mile of pipe	1 440	1 510	1 520
Per tap	17.25	17.50	16.97
Per million gallons	63.00	59.40	54.20
Percentage of gross	65.7%	66.2%	66.8%
Hydrant rental (total)	$153 600	$176 800	$205 200
Per hydrant	40	40	40
Per mile of pipe	380	340	360
Taxes (annual)	84 000	95 200	103 680
Percentage of gross income	8%	8%	8%

Note.—The units have been computed by slide-rule.

It is further assumed that the investment in the physical part of the existing plant amounted to $8 600 000 at time of taking.

The problem, therefore, consists in determining: first, the interest loss during construction, and second, the going value.

Date of Valuation.—The valuation of this plant is assumed to be made as of January 1st, 1910.

Construction Period Required.—It is assumed that a period of 6 years—say from 1910 to 1916—would be reasonably required to build a new comparative plant identical with the existing plant, as of the date of taking in 1910.

Going Value Development Period.—It is assumed that a period of 10 years (or, in other words, 4 years in addition to the period of construction) will be required by the comparative plant to develop an income identical with that of the existing plant.

Beginning of Operation.—In spite of the fact that a 6-year period has been assumed to be required to build the comparative plant, it is assumed that a prudent investor would plan his order of construction so as to make available at the earliest possible moment certain parts of the comparative plant, such as the water supply and distribution pipe system, so that the latter could be put into commission and become a source of revenue at as early a date as practicable. Obviously, the magnitude of the interest charges on the comparative plant, during the construction period, would be a strong incentive to such a course of action.

Character of Plant and Order of Construction.—For purposes of illustration, it is assumed that the plant under question consists of a pond or driven-well supply, of limited extent, supplemented by a filtered river-water supply derived through large impounding or storage reservoirs, involving the construction, not only of a filter plant, but of expensive impounding reservoirs, dam, and conduits, two pumping stations, one to care for the small pond or driven-well supply, the other to care for the main supply from the impounding reservoirs.

The assumed order of construction is shown in Table 2, grouped under:

1. Distribution pipe system;
2. Pond or driven-well supply, and sterilizing plant, with pumping station;
3. Filter plant, conduits, etc.;

4. Pumping station for enlarged supply and extension of same after the year 1913 by the installation of additional pumping units;
5. Storage dam and impounding reservoirs.

The real estate, water rights, and rights of way are assumed to be distributed in the several divisions to which they appertain.

The cost of engineering and contingencies is also included under the several individual items cited above.

The interest during construction, however, as well as the going value, remains yet to be determined.

TABLE 2.—Interest During Construction and Progressive Amount of Investment.

"The Comparative Plant."

Date.	Distribution pipe system.	Pond or driven-well supply, sterilizing plant and pumping station.	Filter plant, conduits, etc.	Pumping station for enlarged supply and extension of same.	Storage dam and impounding reservoir.	Total Net Physical Value, Exclusive of Going Value and Interest During Construction.	
						During year.	To date.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
January 1st, 1910.	$400 000	$400 000	.....	.....	.....	$800 000	$800 000
January 1st, 1911.	900 000	300 000	$300 000	$100 000	$100 000	1 700 000	2 500 000
January 1st, 1912.	900 000	.....	500 000	400 000	300 000	2 100 000	4 600 000
January 1st, 1913.	900 000	.....	300 000	300 000	400 000	1 900 000	6 500 000
January 1st, 1914.	800 000	.....	.....	200 000	300 000	1 300 000	7 800 000
January 1st, 1915.	500 000	.....	.....	100 000	200 000	800 000	8 600 000
January 1st, 1916.							
Totals.....	$4 400 000	$700 000	$1 100 000	$1 100 000	$1 300 000	.....	$8 600 000

Beginning of Operation of Different Portions of Comparative Plant.—It is assumed that by January 1st, 1912, the small supply from pond or driven-wells will be available. The filter plant and main pumping station is assumed to be completed and ready for service as of January 1st, 1914, the additional pumping units installed thereafter being ready for service between January 1st, 1915 and 1916. It is assumed that on or about January 1st, 1915, the storage dam and impounding reservoirs can be put into service, though not finally

completed until January 1st, 1916, and that the pipe distribution system built in any year is available for service the following year.

Loss of Interest During Construction.—In Table 3 has been computed the loss of interest during construction on this plant, based directly on the assumptions made as to the order of construction shown in Table 2 and the discussion following, and on the further assumption that the dividing line for the charges for loss of interest on construction account and on operating account is drawn at the point of completion and putting into service of any revenue-producing unit of the plant.

TABLE 3.—Comparative Plant.
Interest During Construction.

Date.	Total Net Physical Value, Exclusive of Going Value and Interest During Construction.		Average amount on which interest is chargeable for one full year to capital account.	Interest on amounts in Column 4 at 6%.	Total Net Physical Value and Interest during Construction, Exclusive of Going Value.	
	During year.	To date, December 31st.			During year.	To date.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1910-1911	$800 000	$800 000	$400 000	$24 000	$824 000	$824 000
1912	1 700 000	2 500 000	1 650 000	99 000	1 799 000	2 623 000
1913	2 100 000	4 600 000	1 550 000	83 000	2 183 000	4 806 000
1914	1 900 000	6 500 000	2 650 000	159 000	2 059 000	6 865 000
1915	1 300 000	7 800 000	1 450 000	87 000	1 387 000	8 252 000
1916	800 000	8 600 000	500 000	30 000	830 000	9 082 000
Total allowance for lost interest-during-construction item				$482 000		

Thus, loss of interest during construction is allowed on the average amount of investment during the year 1910 at the rate of 6%, on the assumption that the work built during that year cannot be revenue producing until after the year 1911, pending completion of the water-supplying works. Similarly, interest is allowed during the year 1911, not only on the investment during the year 1910, but on the investment during the year 1911. After January 1st, 1912, however, the interest on the investment in pond or driven-well supply is assumed to be charged to operating cost as fixed charges, and not to capital account under the head of interest lost during construction, for the reason that it is assumed that this portion of the plant, being available through the agency of the pond or driven-well supply and the distribution

pipe system built up to that date, is now in the revenue-producing class. Similarly, interest lost during construction is charged to capital account on the filter plant, during the years 1912 and 1913, and thereafter to the operating account as fixed charges, for the reason that the filter plant is assumed to be in revenue-producing operation after January 1st, 1914, when it is assumed to have been completed, although the water-works plant as a whole is not assumed to be finished until January 1st, 1916.

New Construction in Going Value Development Period Ignored.—No consideration is taken of the new construction required during the construction and going value development period, 1910 to 1920, for the reason that, whatever profit or loss is involved thereby accrues alike to both plants in the comparative method, and so has no effect on the going value under determination.

Operations of Existing Plant.

In Table 4 are shown the assumed operating statistics for the existing plant.

TABLE 4.—Existing Plant.

Annual Income and Expense.

Year ending December 31st. (1)	Gross income. (2)	Operation, maintenance, and taxes. (3)	Net income, exclusive of depreciation and fixed charges. (4)	Depreciation. (5)	Net income applicable to fixed charges and dividends. (6)
1910	$1 050 000	$360 000	$690 000	$90 000	$600 000
1911	1 080 000	370 000	710 000	91 000	619 000
1912	1 110 000	380 000	730 000	92 000	638 000
1913	1 140 000	388 000	752 000	93 000	659 000
1914	1 165 000	395 000	770 000	94 000	676 000
1915	1 190 000	402 000	788 000	95 000	693 000
1916	1 213 000	409 000	804 000	96 000	708 000
1917	1 235 000	414 000	821 000	97 000	724 000
1918	1 255 000	420 000	835 000	98 000	737 000
1919	1 275 000	425 000	850 000	99 000	751 000
1920	1 296 000	430 000	866 000	100 000	766 000

Operations of Comparative Plant.—In Table 5 are shown the assumed financial operations of the new comparative hypothetical plant, without allowances for the income from unemployed capital.

Income on Unemployed Capital.—In Table 6 is shown the first trial computation relating to the income on unemployed capital. It will be remembered that it was suggested that, in this method of

computing going value, one should consider himself at the parting of the ways, with an amount of capital in hand sufficient to buy the existing plant—covering in the cost thereof the value of the physical plant, interest during construction, going value, and franchise value—and follow through the steps, relating to investment in the comparative plant, which the capitalist would take, and the return which he would get in the form of interest on his unemployed capital.

TABLE 5.—Comparative Plant.
Annual Income and Expense.
(From Operation, Exclusive of Interest on Unemployed Capital.)

Year ending December 31st.	Gross income.	Operation, maintenance, and taxes.	Depreciation.		Net income applicable to fixed charges and dividends.
			Sum on which it is based.	Amount at 1%.	
(1)	(2)	(3)	(4)	(5)	(6)
1910	.....	.....	.....	.....	.....
1911	$50 000	$118 000	.....	.....	$68 000
1912	195 000	195 000	.....	.....	0
1913	385 000	255 000	$2 123 000	$21 000	109 000
1914	575 000	305 000	3 106 000	31 000	239 000
1915	760 000	344 000	6 065 000	61 000	365 000
1916	923 000	372 000	7 152 000	71 000	480 000
1917	1 063 000	394 000	9 082 000	91 000	578 000
1918	1 175 000	410 000	9 082 000	98 000	667 000
1919	1 250 000	422 000	9 082 000	99 000	729 000
1920	1 296 000	430 000	9 082 000	100 000	766 000

Let it be assumed that he could place idle funds at a 4% rate, except as to the funds required for the year's construction, on which the bank rate of interest is assumed to be 2 per cent. For simplicity, let it be further assumed that on the money expended during one year, 6 months' interest at 2% can be obtained from the bank, corresponding to a 2% rate on bank balances during the year.

Obviously, the first difficulty with which one is confronted is that the going value has not yet been computed. The comparative physical plant was assumed to have a value of $8 600 000; the loss of interest during construction was found, by Table 3, to amount to $482 000; the total investment in physical plant, including loss of interest during construction in the assumed existing plant and hence also in the comparative plant, is therefore $9 000 082. The going value, however, is not yet known, and, therefore, must be determined by a series of approximate computations. The franchise is assumed herein to have nominal value only.

TABLE 6.

First Trial Computation of Net Annual Income (from Operation and from Interest on Unemployed Capital) of Comparative Plant.

Assumed Capital in hands of the Investor-Builder on January 1st, 1910:

Value of physical plant	$8 600 000
Interest during construction (Table 3)	482 000
Assumed Going Value (First Trial)	1 800 000
	$10 882 000

		Determination of Interest on Unemployed Capital of Comparative Plant.								
Year ending December 31st.	Net income from operation (see Column 6, Table 5).	Total investment to date in comparative (physical) plant and interest during construction.	Going value.*	Total assumed investment to date. Column 3 plus Column 4.	Amount invested, which is earning 4% interest.	Amount withdrawn for construction during the year.†	One Year's Interest on		Total interest. Column 8 plus Column 9.	Total annual income or return on comparative plant. Column 2 plus Column 10.
							Column 6 at 4%.	Half of Column 7 at 2%.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1910....	0	$824 000	0	$824 000	$10 058 000	$824 000	$402 000	$8 000	$410 000	$410 000
1911....	$68 000	2 623 000	$179 000	2 802 000	8 080 000	1 978 000	323 000	20 000	343 000	275 000
1912....	0	4 806 000	485 000	5 291 000	5 591 000	2 489 000	224 000	25 000	249 000	249 000
1913....	109 000	6 865 000	811 000	7 676 000	3 206 000	2 385 000	128 000	24 000	152 000	261 000
1914....	239 000	8 252 000	1 126 000	9 378 000	1 504 000	1 702 000	60 000	17 000	77 000	318 000
1915....	365 000	9 082 000	1 395 000	10 477 000	405 000	1 099 000	16 000	11 000	27 000	392 000
1916....	480 000	9 082 000	1 607 000	10 689 000	193 000	212 000	8 000	2 000	10 000	490 000
1917....	578 000	9 082 000	1 752 000	10 834 000	48 000	145 000	2 000	1 000	3 000	581 000
1918....	667 000	9 082 000	1 842 000	10 924 000	— 42 000	90 000	— 2 000	1 060	— 1 000	666 000
1919....	729 000	9 082 000	1 884 000	10 966 000	— 84 000	42 000	— 3 000	1 000	— 2 000	727 000
1920....	766 000	9 082 000	1 897 000	10 979 000	— 97 000	13 000	— 4 000	0	— 4 000	762 000

* These items are determined most easily by carrying the computations involved in Tables 6 and 7 along simultaneously. It is thus assumed that the new plant develops its going value gradually.

† To cover physical plant, interest during construction, and going value.

As a first trial, assume a going value of $1 800 000. Under this assumption, the total value (or cost to the investor) of the existing plant would be as follows:

Physical plant	$8 600 000
Loss of interest during construction	482 000
Assumed going value	1 800 000
Total value	$10 882 000

Therefore, the investor who has concluded to build a new hypothetical plant, instead of to purchase the old plant, is assumed to have in hand the sum of $10 882 000, as of January 1st, 1910. He is assumed, further, to adopt the same order of construction for the new comparative plant as shown in Table 2. Under these assumptions, Tables 6 and 7 result.

TABLE 7.—First Trial Going Value Computation as of January 1st, 1910.

Year ending December 31st.	Total Net Return.			Present Worth of Excess Earnings of Existing over Comparative Plant.			
	Existing plant.	Comparative plant.	Excess of existing plant.	Period of years.	Factor at 6%	Amounts.	Cumulative amounts.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1910	$600 000	$410 000	$190 000	1	0.9434	$179 000	$179 000
1911	619 000	275 000	344 000	2	0.8900	306 000	485 000
1912	638 000	249 000	389 000	3	0.8396	326 000	811 000
1913	659 000	261 000	398 000	4	0.7921	315 000	1 126 000
1914	676 000	316 000	360 000	5	0.7473	269 000	1 395 000
1915	693 000	392 000	301 000	6	0.7050	212 000	1 607 000
1916	708 000	490 000	218 000	7	0.6651	145 000	1 752 000
1917	724 000	581 000	143 000	8	0.6274	90 000	1 842 000
1918	737 000	666 000	71 000	9	0.5919	42 000	1 884 000
1919	751 000	727 000	24 000	10	0.5584	13 000	1 897 000
1920	766 000	762 000	4 000	11	.....	2 000	1 899 000
					Resulting Going Value		$1 899 000
					Assumed Going Value		1 800 000
					Error on First Trial Computation		$99 000

Resulting Going Value under First Trial Computation.—In Table 7 is shown the result of the first trial computation of going value. It will be noted therein that the going value, which was assumed to

be $1 800 000, was found to be approximately $1 899 000, involving an error in assumption of $99 000.

Were a going value of $1 900 000 assumed in the second trial computation, the resulting going value would be found to be somewhat less than that sum, on account of the greater amount of interest accruing on the unemployed capital, which tends to reduce the going value, the other items remaining substantially the same.

Probably closer results would be obtained by assuming for the second trial computation a going value of $1 880 000.

Recomputation Unnecessary for the Purpose of This Paper.—As going through the somewhat laborious process of recomputing Tables 6 and 7 would not shed any additional light on the principles enunciated, it is unnecessary, for the purpose of this discussion, to follow through this recomputation. Enough has been said to illustrate the principles involved by the method of computing going value suggested by the writers.

For the second trial, assume a going value of $1 880 000. The total value of the plant under discussion would then be:

Physical plant	$8 600 000
Interest during construction	482 000
Assumed Going Value (second trial)	1 880 000
Total value	$10 962 000

In the second trial computation, therefore, it should be assumed that the builder of the comparative plant starts, as of January 1st, 1910, with the sum of $10 962 000 in hand.

Methods Identical when Franchise Value is Included.—Attention may be called to the fact that no allowance has been made for franchise value, on the assumption that the franchise has nominal value only, in the works under question, and that, under the forms of franchise enjoyed by many works, and with the present-day tendency of the Courts and public service commissions, water-works are approaching a condition where they may be limited in earnings to an amount which will pay dividends substantially only on the net physical and going value of the property. This has the effect of reducing franchise value to the mere right to do business.

If, on the other hand, the plant under question had been assumed

to have a substantial franchise value, as is the case in many works, this franchise value should have been added to the sum of the physical value, plus interest during construction, plus going value, to make up the amount of the unemployed capital which the investor is assumed to have in hand, before making his comparison between the existing works and a hypothetical works.

The preliminary costs of promotion, organization, etc., and other kindred items, are assumed to have been included in the physical valuation.

Effect of Assumptions as to Order of Construction on Loss of Interest during Construction Item.—It will be apparent, from the foregoing discussion, that the larger the allowance for the loss of interest during construction item, the smaller will be the resulting going value, for the reason that, the gross income and operating expenses remaining the same, the amount of unemployed capital in the hands of the builder, will be increased. This, in turn, increases the interest accretions on unemployed capital, which have to be deducted from the net income of the existing plant in determining going value. The effect, therefore, of these deductions is to decrease the going value. It is obvious, however, that the decrease in going value is not equal to the increase in interest charges, for the reason that the interest allowances on unemployed capital are assumed at a lower rate than the loss of interest during construction item, and the present worths of the annual differences in net income are taken in determining going value as of any given date, instead of the arithmetical sum of those differences.

Review of Going Value.

Having determined the going value of any works by the comparative method, it remains to review the matter in a broad way, for it must be remembered that cost is not always synonymous with value.

Thus, on the plant assumed herein, the resulting going value of $1 880 000 amounts to:

20.7% of its estimated net reproduction cost, meaning thereby the reproduction cost less accrued depreciation to date of taking, including the early organization expenses, engineering and contingencies, interest during construction, etc.;

17.1% of the total net reproduction cost, including in the latter the reproduction cost of the business of the company, or, in other words, the going value;

1.8 times the amount of the gross annual income of the old plant for the fiscal year ending January 1st, 1910;

3.1 times the amount of the net income, as of January 1st, 1910, meaning by "the net income" the difference between gross income and the sum of the operating expenses, maintenance, taxes, and depreciation.

It remains then to consider whether the plant is, at the moment:

(1) "Over-built"; that is, at the end of a reconstruction period, so far as the physical plant is concerned, and at the beginning of a recuperative period, so far as its financial condition is concerned, inasmuch as some of the construction work has been built for future rather than present needs, and the rates are therefore at the moment incommensurate with the character of the service being furnished;

(2) "In a normal stage of development" where the rates may be said, broadly speaking, to be commensurate with the character of the service furnished;

(3) "Under-built," that is, where the plant is outgrown, or where it is collecting rates which are excessive for the character of the service being furnished by it, in which case the plant may be said to be rapidly approaching a reconstruction or recuperative period physically and a reformative period financially.

Effect of Size of Plant.—It is obvious that, in the case of a small plant, the construction of which can be executed in one season's work, no such complication in regard to the interest on unemployed capital nor necessity for refinement in computation exists.

The writers have felt that it might be of interest, however, to call attention to the difference in the nature of the problem when dealing with a large plant requiring a period of several years for its construction, and to point out a method for the determination of the going value in such cases.

General Applicability of Principles.

The principles enunciated herein, if sound, should find general applicability in the valuation of all forms of business enterprises. Good will in general business enterprises should be susceptible of similar analysis to that herein outlined for public service corporations, when the theory of reproduction is utilized as the basis of appraisal, provided due weight be given to the modifying conditions or circumstances under which the property under valuation is operating.

Recapitulation.

Going value is defined as the value of a created income. While it pertains to the business rather than to the physical plant, it is nevertheless true that in any just determination of the reproduction cost of a property it is as real an element in this reproduction cost as is the cost of reproducing pipe lines, pumping stations, or any other part of the physical plant.

As a means of determining going value, the "comparative method" is suggested. This consists in an analysis of the relative net earnings or return to be derived from the existing plant as compared with that from a hypothetical "comparative plant" which is assumed to be built on the date of valuation and to acquire the business of the existing plant, in the territory served by it, as rapidly as possible, under non-competitive conditions. The sum of the present worths of the annual excess in net earnings, or return, from the existing plant, as compared with those from the comparative plant, in the period of years from the date of valuation to the time when the earnings of the comparative plant can reasonably be assumed to equal those of the existing plant, is then the measure of the going value of the existing plant. It represents the amount which a purchaser could afford to pay for the existing property with its established income, in excess of the value of its bare physical plant.

In analyzing the going value of any public service corporation, it is suggested that the investigator place himself in the position of an investor having in hand the necessary capital, either to buy the existing going plant with its established business, or to build a new comparative plant to replace it and its established business under non-competitive conditions, and then determine the return which he would receive upon his capital in both cases.

The principal factors affecting going value are:

1. The period of construction required for building the comparative plant;
2. The going value development period;
3. The rate of acquisition of income by the comparative plant;
4. The stage of development of the existing plant, whether over-built, normally developed, or under-built;
5. The difference in annual depreciation charges for the two plants during the going value development period;
6. The rate of return assumed upon the unemployed capital of the builder, up to the time of completion of the comparative plant.

The application of these principles is illustrated in the solution of a hypothetical problem, involving the determination of the going value of a water plant in a city assumed to have 300 000 population.

In the case of small plants, requiring but a year, more or less, for construction, all the refinements suggested in this paper are not necessary.

The principles outlined in the "comparative method" suggested, if sound, would seem to settle the controversy which has arisen as to the propriety of including interest or fixed charges as an operating expense. Clearly, if the principles enunciated are sound, the fixed charges should not be so included.

In conclusion, the writers dissent from the view taken by the Wisconsin Public Service Commission as to the nature and method of computing going value in determining the reproduction cost of a public service property.

DISCUSSION

Mr. Burns.

CLINTON S. BURNS, M. AM. SOC. C. E. (by letter).—The writer wishes to express his appreciation of the opportunity to read a paper so well prepared by men who have devoted much time and study to this comparatively new field of appraisement work—"Going Value."

It has been the writer's privilege to be associated with one of the authors in a number of appraisals of water-works properties, and he is fully in accord with the general views expressed in this paper, but in some particulars he must take issue, though it is with reluctance that he ventures to enter a discussion which may be somewhat at variance with the opinions of men who have so ably presented their subject. This discussion, however, is not to be construed as criticizing, or detracting from the merit of, the paper, but rather to add to its value, if possible, by pointing out some instances wherein its logic may be strengthened, or minor discrepancies may be eradicated.

The writer fully agrees with the authors in that going concern cost is as much a part of the reproduction cost of a public service corporation's property as is the reproduction cost of its physical plant. He is fully in accord with the statement that in reproducing a plant it is only proper and logical to consider what it would cost to rebuild it under present conditions; and that this applies to the business of the corporation as well as to its physical property. The writer likewise uses the method adopted by the authors, which they term the "Comparative Method."

A Problem in Futures.—It should be pointed out, however, that the determination of going value is a problem of the future, and therefore past or present income cannot be properly used as a basis of computation, except in so far as the past or present may be taken as a fair criterion of the future. The writer maintains, therefore, that the only basis on which going value may be computed, with absolute impartiality and fairness to both parties, is to determine first what is a fair rate in the aggregate, or a proper return on the fair value of the property, and then apply the comparative method to determine the going value. If the present income of a property is unreasonably low, due to rates which are insufficient, it is manifestly unfair to the company or corporation to compute the going value on that income; and if the present income is unreasonably high, it is equally unjust to the municipality to use that income as a basis of computation; furthermore, it is purely speculative to assume that the income will continue to be unreasonably high in the future, especially in view of the ever-increasing tendency toward the regulation of public service corporation rates. Therefore it is questionable whether the authors' definition of going value—"the cost of acquiring a given

income"—is a logical one, unless it be first determined that the "given income" is a reasonable one, or a fair return on the fair value of the property. The fair value referred to must, of course, include the going value as well as the physical value. Mr. Burns.

Definition of Going Value.—In a discussion of the paper by H. E. Riggs, M. Am. Soc. C. E., entitled "Valuation of Public Service Property,"* the writer defined "Going Value" as:

"The present worth of the amount by which the anticipated profits of a going plant, operating at reasonable rates, exceed the present worth of the anticipated profits of a similar hypothetical starting plant [the comparative plant], operating at those same rates."

Further study seems to strengthen his faith in this definition as the logical conception of going value, as it seems especially well adapted to meet the requirement of fairness to both parties. If any public service corporation takes exception to it, it must thereby concede that its rates are unreasonably high. If the municipality takes exception, it thereby acknowledges that the rates are too low, or else confesses its desire to be unfair to the corporation.

At the Parting of the Ways.—The authors' illustration of the assumed capitalist, at the parting of the ways, is well presented, and, if their reasoning be pursued to its logical conclusion, there should be no confusion of ideas in the resulting "Going Value." They state, further, that this assumed capitalist, should he take the first path, and buy the existing plant,

"will evidently receive as return on his investment the net income indicated by its past history, modified by such circumstances as will be likely to influence its net income during the period when a new comparative plant can reasonably hope to overtake its development."

This statement certainly must contemplate that there may be, and in fact are sure to be, some circumstances which will influence the net income of the future and may not be apparent from the past history of the property. Paramount among these circumstances is rate regulation, and, this being true, no prophecy of future net income is based on sound logic unless the possibility of rate regulation be considered. Consequently, the capitalist at the parting of the ways will certainly have cause to complain at his engineer's lack of judgment if this factor be overlooked in his computation of going value.

The Rate of Interest on Unemployed Capital.—The authors' statement, that "the larger the rate of interest allowed on the unemployed capital, the smaller the resulting going value," is apparently correct logic. However, the corollary of this statement would be as follows: If the unemployed capital is invested at a rate of interest equal to the net return from the existing water plant, then the going value reduces

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 1.

Mr. Burns.

to zero. This phase of the problem merely emphasizes the correctness of the writer's contention that the income must be that due to reasonable rates. For it is manifest that, if the net income of the going plant is not greater than the return on unemployed capital, then the rates, in the aggregate, are too low, for no company should be required to perform a public service for less compensation than a proper return on capital invested in other pursuits involving the same degree of risk; and capital invested is entitled to larger returns than liquid capital.

It must be remembered that capital in liquid form produces no returns *per se* until invested in some productive enterprise, although the possessor of such capital may allow some bank or other investor to use it temporarily, keeping it in easily convertible invested form, and for this permission the bank or other investor pays to the possessor a portion of the profits accruing to the temporary investment, thus resulting in a low rate of interest.

Going Value Development Period.—The authors state that the past history of water-works, many of which were built during the Eighties, is not a fair criterion for the period required for the acquisition or development of business by water-works to-day. This appears to be in conflict with their prior statement on the same subject, but this latter statement is perhaps the result of more careful consideration and a deeper study of the subject.

The reason offered for this statement, that the public is now educated to higher standards of living, and to the use of water under pressure, which would result in much more rapid development of income by water-works now than formerly, is undoubtedly true; but this is by no means the principal reason. Of far greater influence is the fact that property is invested in house fixtures, plumbing, and in sewer connections.

Some experts maintain that were it not for the past existence of the water-works system, none of these vested property interests would be in existence, and that therefore these must be ignored in estimating the length of time required for the acquirement of the business. This, however, seems to be an assumption entirely contrary to pure logic, and not warranted by the facts. A corporation cannot take control of the vested property of an individual, nor claim any assets or benefits accruing therefrom. Hence, when considering the comparative plant, all property other than that belonging to the water company itself must be assumed to remain intact, and any computation of going value that fails to take these conditions into account is not in accord with correct logic. These plumbing fixtures, house connections, bath-rooms, sewer connections, lawns that demand water service, and all kindred metropolitan conditions that compel the uninterrupted continuance of the water service, are factors the existence of which cannot be denied. They are present actualities, and just as much to be considered as any

local factor affecting the cost of reproduction of any of the physical property. Mr. Burns.

If, then, the appraiser is consistent in his logic, he must face these facts and give due consideration thereto in his estimate of going value, just as faithfully as he gives consideration to the local factors which affect the cost of duplication of any of the physical property. None of the theories advanced by the authors seems to cover the true situation, or to take into account all these factors so vital to a correct and strictly logical solution of the problem under discussion.

The Stage of Development.—Whether the plant finds itself in a stage of development termed "over-built," "normal," or "under-built," is recognized by the authors as a proper subject of inquiry in determining the going value. They state that the ability of the over-built plant to grow in income to its new capitalization cannot be recognized in computation of going value; but, in the case of the under-built plant, they state that the net income may be substantially reduced by the expenses incident to the additional capital required to bring the plant to a rational standard of water-works construction. This latter statement, when applied to their computation of going value, would, of course, substantially reduce it.

If it is true that the under-built plant should be brought to a rational standard, and that the computation to determine going value takes this fact into account by presupposing the normal condition to exist, then why not also treat the over-built plant similarly? Would not any other treatment be inconsistent with the logic? This again emphasizes the correctness of the writer's contention, that going value computation should be based on the theoretical net income which would accrue from reasonable rates, regardless of actual present income. Present income may be unreasonably high, or so low as to border on confiscation, and in neither of these situations would it be fair or equitable to use actual income as the proper basis of computation. On the other hand, if a theoretically reasonable net income be assumed as a foundation for computation, the question of under-built or over-built condition is automatically adjusted in determining the proper net income to assume in the particular case under consideration.

Interest During Construction.—The authors have evidently fallen into error in their statement:

"Interest-during-construction payments are inevitable in all projects; * * * they constitute a forced expenditure, alike with the cost of the bricks and mortar, the pipes and the pumps, and cannot be recovered in any way except by addition to capital account or floating debt."

It is a little surprising that the authors should have become thus confused on the very point which they had just stated had been the source of some confusion in the minds of those who approach the sub-

Mr. Burns.

ject for the first time. That they are in error is manifest by a moment's consideration of their original hypothesis of the assumed capitalist, at the parting of the ways, "having in hand the necessary capital." Now, a man who has the capital in hand, seeking an opportunity to convert it from liquid form to invested form, does not have any interest charges to pay during construction. He is constructing solely to provide a means whereby his idle capital may become useful and bring him returns. To make this point more evident, suppose that the assumed capitalist had in hand the necessary bricks, mortar, pipes, and pumps, and the necessary labor with which to construct the plant, instead of having in hand the necessary capital. By putting his idle labor at work and utilizing his otherwise useless materials, he may convert his property into a water-works plant which will bring him returns, but in so doing he does not incur any item of cost for lost interest during construction. Neither does the capitalist having in hand the necessary capital. Both are in exactly similar positions, seeking a means whereby interest may be earned on their idle property.

The lost interest during construction, therefore, must of necessity appear as an element of the going value, by the comparative method, and to include it also in the capital account charged against construction of the physical plant is a duplication of accounts. If their original hypothesis had assumed a penniless promoter, at the parting of the ways, with ability to borrow the necessary capital, lost interest during construction would then be a proper item to charge in the capital account of the comparative plant, but in this event the going plant would also have interest chargeable against it. The error seems to be that they have treated the comparative plant as having to be constructed by the penniless promoter, then this promoter is miraculously converted into a capitalist, without having bridged the chasm between promoter and capitalist—popularly supposed to require a bridge of no mean proportions.

Application of Principles.—This duplication of accounts is clearly shown in Table 3, wherein $482 000 is stated as lost interest during construction, while Table 4 shows the total net profits of the existing plant. The capitalist at the parting of the ways, with capital in hand, had he taken the first path and purchased the existing plant, would have come into the immediate enjoyment of the net returns set forth in Table 4; but, had he taken the other path and elected to construct the "comparative plant" he would not have lost this $482 000, designated as interest lost during construction, but instead thereof he would have lost the opportunity to realize the greater net return shown in Table 4. He could not include any of this $482 000 as an item of expense in the capital account of the comparative plant, because, by the fundamental hypothesis of the comparative method, his capital

is not earning, but is seeking the opportunity to earn. He could not lose interest that he did not possess. Mr. Burns.

Conclusion.—Going value is a subject which has but recently received scientific study and careful consideration from engineers and appraisers. Any method advocated for its determination, therefore, must of necessity pass through the successive stages of development so essential to the perfection of any new subject, and it is only by earnest study, honest effort, and fair-minded discussion of such papers as this, that true progress can be attained. It is largely an exercise in pure logic, and, therefore, it is not surprising that some erroneous conclusions should have been developed, based perhaps on theories springing from illogical assumptions. It is with a keen appreciation of these conditions that the writer offers this discussion on a subject second to none in its demands for clear thought, deep logic, and correct conclusions.

Any theories advanced by the writer are offered to invite discussion, in the hope that this much complicated subject may thereby be freed from some of the confusion with which it has become enveloped.

Albert I. Frye, M. Am. Soc. C. E. (by letter).—This able paper brings out a new point to be considered in connection with the valuation of water-works, namely: Shall the going value be considered as the actual going cost, economically administered, in building up the business of the existing plant? or shall it be considered as the estimated going cost necessary in developing the business of the prospective duplicate plant? Mr. Frye.

The writer believes that before a fair valuation of the works can be made, the appraisers should gather and formulate the following data:

Aa. The actual cost of the existing physical plant;
Ab. The actual going cost in connection with *Aa;*
Ba. The estimated cost of the duplicate physical plant;
Bb. The estimated going cost in connection with *Ba;*
K. A curve or table showing the annual profits, in percentages, on the total investments.

Under normal conditions of good management, the above going costs may be considered as going values; otherwise they may be reduced to going values by possible elimination of excessive or unnecessary items.

The basic cost of the project would then be considered to be either, $C = Aa + Ab$; or, $C_1 = Ba + Bb$; or, some compromise between the two; and the appraised value of the project should be based on all three of the items: $V = Aa + Ab$; and, $V_1 = Ba + Bb$; and, $V_2 =$ the average of the last few years' profits capitalized at the usual industrial rate of interest, say 6%, more or less.

Mr. Frye. These should be weighed and balanced very carefully, and it is of course understood that interest is to be included in each of the above cases.

The contention of the authors that the estimated going cost, *Bb*, rather than the actual going cost, *Ab*, should be associated with the estimated cost, *Ba*, in arriving at a fair appraisal value, is, in the writer's opinion, a tenable one.

Mr. Stearns. FREDERIC P. STEARNS, PAST-PRESIDENT, AM. SOC. C. E. (by letter).—At the present time, the growing prominence of matters relating to the valuation and rate regulation of public service properties warrants a full discussion by engineers of all questions relating thereto, and the authors of this paper have presented a valuable contribution on the subject.

It may be well to state in the beginning that the writer has been connected with the valuation of water-works property for the purposes of sale and rate fixing, and although the principles involved in such valuations are in the main the same as in those of other public service properties, they necessarily differ in detail.

The term "going value," or "going concern value," is an unfortunate one, because it may be given different definitions. The authors have defined it "as the value of a created income, or—from the reproduction point of view—the cost of acquiring a given income." They also speak of it as "the investment necessary to put the plant into successful operation and to create the revenues that justify its construction," and, after this statement, say: "This portion of the cost of a project has been termed by the Courts, the 'going value,' or the 'going concern value.'"

In many cases, engineers and the Courts have referred to the "going concern value" in such a way as to show clearly that they do not have in mind the definitions above given; for instance, Justice Moody, of the Supreme Court of the United States, in the Knoxville case, referring to an allowance for the "going concern," says:

"The latter sum we understand to be an expression of the added value of the plant as a whole over the sum of the values of its component parts, which is attached to it because it is in active and successful operation and earning a return."

The Judge of a United States Circuit Court in California, in rendering an important decision, less than three years ago, said:

"The value of the franchise and going business depends upon their earning power."

He also said:

"A plant with an established business, with customers who have connected their houses with the Company's distributing pipes is more valuable than it would be without such connections and without such customers."

When there is competition, there may be a "good-will" value connected with a going concern which would not exist if the concern were not a going one, and this part of the value of the going concern would be based on the earnings and the degree of success which has been attained; but when the public service corporation has a practical monopoly and the commodity sold is an absolute necessity—as, for instance, water to the inhabitants of a large city—it seems clear that, under such circumstances, there can be practically no "good-will" value to be included in the "going value." Mr. Stearns.

This is the view expressed by the United States Supreme Court in the Consolidated Gas Company case, where it stated:

"We are also of opinion that it is not a case for a valuation of 'good will.' * * * The complainant has a monopoly, in fact, and a consumer must take gas from it or go without. He will resort to the 'old stand,' because he cannot get gas anywhere else. The court below excluded that item, and we concur in that action."

The writer believes it would aid the Courts and others if a new term were used for that portion of the going value which represents the special expenses and risks connected with the construction of the works, and the expense of developing the business of the original works and of additions thereto. The propriety of including in this portion of the going value other sums than those relating to the development of the business will be discussed later.

Halbert P. Gillette, M. Am. Soc. C. E., in his recent discussion of Mr. Riggs' paper, "The Valuation of Public Service Corporation Property,"* has used the term, "development expense," in the same sense as the authors have used the term, "going value," and this or some similar term seems less likely to cause confusion than the term, "going value," which, under some circumstances, has a more comprehensive meaning.

It seems to be generally admitted that the Courts favor the reproduction method of ascertaining the physical value of property, and that they do this, especially in the case of old property, first, because there may be marked changes in the value of real estate and the cost of works from the time they were built to the time when they are valued, and it is the present and not the past value that is desired; second, because of the difficulty of proving the legitimate cost of works constructed many years before, on account of defective bookkeeping and illegitimate and extravagant methods of construction; and third, on account of the abandonment of parts of the work which were included in the original cost.

There is no reason to believe that, with normal works constructed in a legitimate manner, with efficient bookkeeping, and without change in the prices of real estate, labor, and materials, the Courts would

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 1.

Mr. Stearns.

prefer the estimated to the actual cost of the works, and it also seems reasonable to believe that they would not hesitate to include the total outgo for the construction of the works and the development of the business, even though there were included in the accounts the cost of a reasonable proportion of temporary and unsuccessful works which had been abandoned and therefore would not appear in the schedule used in determining the cost of reproduction.

The writer, therefore, believes that, in determining the reproduction value of the tangible and intangible elements connected with the development of normal works and of the business of such works, the engineer should endeavor to obtain results agreeing as closely as possible with the probable outlay if the works in question had been built and the business developed when the value of real estate, the prices for labor and materials, and other circumstances, were the same as at the time of valuation.

Assuming the above view to be correct, does the authors' method of ascertaining the "going concern value," defined as the cost of developing the business, represent the probable outlay for such development under similar circumstances as to prices and otherwise?

Applying the rule to water-works, it is the writer's judgment, that, in the case of the works first introduced for the general supply to a city or town, it does apply, and gives a development expense which would be in fair accord with the actual outlay for developing the business.

He is as firmly of the opinion that, in the case of the water supply of a large city which long ago outgrew the capacity of its original works and is supplied mainly by works which have been added from time to time, the method used by the authors will give a result wholly out of accord with the actual outlay for developing the business of such works, even after making due allowance for difference in prices and in other conditions at the time when the outlay was actually made and at the time of valuation.

It is clear that when works are first introduced into a city or town where the water supply is obtained from wells, by inhabitants unaccustomed to a public water supply, and where for several years the demand for water is likely to represent but a small part of the capacity of the original works, the cost of developing the business is likely to be a considerable percentage of the cost of the works; whereas the cost of further developing the business to the extent required to provide a return on expenditures for additions to the works will generally be a much smaller percentage of the cost of such additions. The latter statement will be especially true in cases where the return which the public service corporation is permitted to collect is based on the physical valuation of the property and is determined at frequent intervals.

Mr. Stearns.

For instance, the provisions of the Constitution of the State of California require that the rates or compensation to be collected for water supplied to any city or town shall be fixed annually by the governing body of such city or town. In fixing such compensation, the physical valuation of the property is generally a factor, and, in accordance with certain legal decisions in that State, it is usual to exclude items of property under construction up to the time when they are put in use, and then to include them as a portion of the operating plant. Additions to a normal plant, therefore, become productive property on which the same rate of return is allowed as on the other property at the time of the next rate fixing. That is to say, the cost of developing the business for the additions to the works is substantially the loss of interest on their cost for a period, averaging less than a year, extending from the time when they are put in use to the time when they are included in the physical valuation.

Nothing in the authors' description of their method of determining the "going value" or "development expense," or in the example illustrating their method, indicates that the results obtained in the case of a large and old plant will be any smaller percentage of the value of the property than in the case of a small plant, although, in accordance with the views set forth by the writer, the cost of developing the business for the additions which make up the greater part of a large plant is a much smaller percentage than for the original plant.

Mr. Alvord has given a table* containing certain data of the valuations of water-works properties, including the going value attributed to such properties. If one takes the data from this table, omitting the places having less than 10 000 inhabitants and the one place having a population of more than 100 000—the former being omitted because of their small size and the latter because it is hardly possible that it could be supplied from its original works—and also omits one case where there was competition between two rival plants, the following minimum, average, and maximum results are obtained:

Average population		34 555
Average net value of physical property		$774 567
Average going value		$133 269
Going value per capita:	Minimum	$1.67
	Average	3.86
	Maximum	6.90
Percentage of value of net physical property represented by the going value:	Minimum	10.5
	Average	17.2
	Maximum	34.6

Mr. Alvord calls attention to the lack of relation between the going value and the value of the physical property, and yet an average

* *Proceedings*, American Water-Works Association, 1909, p. 206.

Mr. Stearns.

of the results attained in a number of cases where rational methods were used will give approximately this relation in the case of a normal plant.

In the typical illustration used by Messrs. Metcalf and Alvord, the statistics are as follows:

Population	300 000
Net value of physical property, including interest during construction	$9 082 000
Going value	$1 897 000
Going value per capita	$6.32
Percentage of value of net physical property represented by the going value	20.9

The writer can easily conceive that there may be an actual deficiency in the returns during the earlier years of the operation of a normal water-works plant which will equal $133 000 for a plant valued at $774 000 and supplying 34 000 people, but he cannot conceive that, during the growth of such a plant to a net value of $9 082 000, with ability to supply 300 000 people, there can be an added deficit amounting to $1 764 000, or any sum approximating this amount, due to the development of the business incidental to the additions made from time to time.

As already stated, he believes that no method will be satisfactory that does not produce results which are in fair agreement with what actually happens in the operation and construction of works. The assumption that a large city, say one having a population of from 100 000 to 1 000 000, is without a water supply and that a new supply is to be introduced, is one which has no parallel in actual practice, and as the results based on this assumption differ greatly from those of actual experience on works which have been developed to supply these large cities, the results for such populations are unsatisfactory.

The writer has already stated that, in addition to the expense of developing the business, there are other expenses and risks in actual practice for which the owner of public service property should be compensated, and which do not appear in the ordinary reproduction value. This view may best be explained by giving some instances.

If one were to value the Quebec Bridge after it has been completed and successfully tested, he would not think it proper to ascertain the cost of the bridge which failed and add it to that of the successful bridge, but, in the writer's opinion, it would be proper to add to the cost of the successful bridge a sum representing the value of the risk incurred in building a work of such unusual magnitude. In other words, he believes that if the company which is now building this bridge could know in advance exactly what it would cost for construction, engineering, interest during construction, and all other items of expense, it would willingly pay a considerably

larger sum for the bridge constructed and successfully tested and of assured stability. Mr. Stearns.

Take another instance, such as the building of a large earth dam having a height of 100 ft. or more. Such dams are frequently built by first constructing a tunnel through an adjoining hill, or by laying a pipe or conduit through the dam, to take care of the flow of the stream during construction. The capacity of such conduits is generally too limited to carry the discharge of large floods. Dependence, therefore, is placed on building the base of the dam during the dry season and on utilizing the storage to assist the conduit in taking care of the floods. In such construction, however, there is a risk that an exceptional flood may come, after the work is well advanced, and destroy it. On the other hand, all may go well until the completion of the work, in which case the dam, in the writer's opinion, is worth more than the cost of construction, by the value of the risk incurred in connection with its construction.

In one case where the writer was engaged with others in the valuation of a masonry dam 222 ft. high, it was originally planned to build a rock-fill dam, and its construction proceeded until it had been built up 54 ft. above the level of the stream, when an exceptional flood removed the whole of the rock-fill in a single night.

The rock-fill dam necessarily did not appear in the schedule of property valued, but, had it been completed and proved successful, it would have been worth more than its cost, on account of the risk incurred in its construction.

Accounts are frequently seen in engineering papers, or are the results of personal observation, where dams and other structures have failed to stand the test of use, or where incidents connected with the construction of works have caused an expense far beyond what is provided for under the head of contingencies.

As a few instances, the writer will cite the slip at the Necaxa Dam, the encountering of bad ground in the Loetschberg Tunnel, in Switzerland, which required the abandoning of about a mile of the tunnel and the diversion of the line through another portion of the mountain, and the change of plan of the new Croton Dam of the City of New York, which involved an expense of more than $1 000 000 for construction and interest, which would not appear in the reproduction value of that property.

These examples might be extended in many directions, to show that public service corporations and cities, without negligence on their part, may expend large sums for works which fail or otherwise are unsuccessful, and for which they will not receive credit in the reproduction value determined in the usual way.

It may be questioned by some whether failures and unsuccessful works are not covered by the allowance for contingencies, which is

Mr. Stearns.

generally included in the reproduction value of the physical property, and must be included in any preliminary estimate of the cost of work to be done, if the engineer intends to have his estimate agree with the probable cost of the completed work. There is no doubt that a sufficient allowance for contingencies would cover also the risk of failures and non-success, but the writer does not believe that it is the custom of engineers to make sufficient allowance for this purpose in works of a hazardous nature.

It would not be fair to the public to charge to the value of the works the amount of money lost by some great disaster, but, on the other hand, it is not fair to the public service corporation constructing hazardous works to charge only the cost of reproducing such works, without adding to such cost the value of the risk which they have incurred. This value the writer would include as a "development expense" and, consequently, as a part of the "going concern value."

In the early years of any large plant, defects in design or apparatus are frequently discovered, and are made good at a considerable expense. Temporary works are frequently installed, and are given up as soon as the permanent works are completed. All such items of expense do not appear in the reproduction value of the physical property, and, unless otherwise provided for, should be included as a part of the "development expense."

Mr. Kuichling.

E. KUICHLING, M. AM. SOC. C. E.—The authors have rendered a service of the greatest value by their clear exposition of an important but intricate subject. There are, however, a few directions in which they might extend their remarks with advantage to those who are called on to make valuations.

The paper appears to assume that the property under consideration is in a prosperous or fully solvent condition. Two other cases are sometimes encountered, the first being the rare one when the property has been highly profitable from the outset, so that the dividends have amounted to much more than the going value computed in the manner indicated; and the second when the actual commercial or market value of the property at a given time is considerably less than the sum expended for construction.

In the first case some writers have urged that the past large receipts have extinguished any claim to more than the estimated cost of reproduction less depreciation, provided the contract by which the highly remunerative rates were fixed has expired. This plea is based on the theory that a corporation which performs a public service under contract is not justly entitled to a profit much in excess of interest, depreciation, operation, and tax charges, and that the initial risk of the security of the investment must be eliminated, because the bulk of the funds for construction are usually furnished by non-resident bondholders who assume all such risk, instead of the operating

company which is formed by the stockholders. It practically seeks to nullify the contract on the ground that the specified rates were exorbitant and oppressive to the public. An argument of this nature is seldom sustained by the Courts when the contract was obtained in a lawful manner. Mr. Kuichling

On the other hand, it can be said that the accumulated profit was legitimately earned under the contract, that the rates seemed fair at the outset, and that the responsibility for them rests on the public officers who executed the contract, hence that the past profits should be left out of consideration entirely. This argument is similiar to that used in the Court's direction to make the valuation on the basis of present prices of materials and labor, instead of the actual cost, on the ground that if the present prices are higher than those which prevailed during the construction of the plant, then the people who are seeking to buy the property should pay the difference, and *vice versa*. It is the argument which is commonly adopted, and it may be said, therefore, that, in the first case mentioned, the going value of the plant is a legitimate asset of the company, and that it should be computed in the manner indicated by the authors.

The second case, however, presents some difficulties, as it is assumed that the revenue is not sufficient to cover the charges for interest, depreciation, operation, and taxes, to say nothing of profit. Sometimes such plants have the appearance of being prosperous by the omission of a reserve fund for depreciation and the payment of the amount which should properly be laid aside for this purpose under the name of dividends. In this financial state the company is losing money, even under good management and after having acquired all the avilable business. If the plant were offered for sale to individuals, it would probably not bring as much as the cost of construction. This may be due to inadequate rates, or to the high prices paid for materials and labor, or to the costs of accidents, such as the loss of a storage reservoir, the wrecking of an expensive pumping engine, the forced abandonment of the source of supply, etc. It may also have become necessary, in the course of time, to purify the water by filtration long before the expiration of the contract, thereby causing a large additional outlay of capital.

Under such conditions, the property has obviously no substantial going value which would be recognized by a private purchaser, but only a potential or prospective one. Before it can attain a value greater than the cost of reproduction less depreciation, either the prices of materials and labor must rise to their former magnitude, or the contract must be modified so as to increase the rates. The latter is often a troublesome matter, and it usually happens that the owners of the property are compelled to suffer the annual deficit until the expiration of the contract. When this time arrives, the past losses

Mr. Kuichling.

will have to be taken into account in estimating the going value under a favorable contract.

When serious losses of capital have been sustained by accidents, or by necessary enlargments of delivering apparatus long before the original apparatus was worn out, the introduction of these sums of money into the going value account is often contested on the ground that the purchaser should not be required to pay for something which no longer exists or which is of no use. This contention is clearly wrong if the plant was originally built in a proper manner and with due regard to the probable future growth of population and consumption. A large increase sometimes occurs suddenly by the establishment of a great local industry, and cannot be foreseen. The water-works plant must then either increase its capacity correspondingly, or it will be driven out of business by its failure to meet the new demands. In the hope that this industrial development will be permanent, the water company makes a large additional investment and abandons much of its former plant. The established rates for the use of the water, however, are too low to compensate the company adequately, and the deficits must be carried along until a better rate schedule can be fixed. The original outlay for the abandoned portions of the plant, as well as the accumulated annual deficiencies in revenue, certainly represent the cost of establishing the business of the water company at the time when it begins to be profitable.

The element of going value is affected in great degree by the nature of the purchaser of the property. If it is bought by a private party, the price paid will manifestly depend on the length of time during which the existing inadequate rates must remain in force under the contract, and also on the probable difficulty of securing favorable rates subsequently; but, if it is bought by the municipality on the expiration of the contract, or even prior thereto, the owners can justly claim reimbursement for their past deficiencies of revenue while they have served the public faithfully, inasmuch as the municipality will presumably increase the rates for the use of the water at once in order to avoid similar loss. This larger prospective revenue will then cause the former potential going value to become substantial, because it represents the cost of acquiring the increased income.

In municipal water-works there is no contract as to rates for a long term of years, and the revenue can easily be increased at any time to conform to new conditions, or the deficits can be covered by general taxation. Little information regarding these matters is found in the annual reports, but, on close examination of the records, it will be found that the aggregate deficits are often surprisingly large. In such cases this has been due to a disinclination on the part of the municipal authorities to increase the rates sufficiently to make the enterprise self-supporting. Municipal works thus have a great ap-

parent advantage over private enterprises for public service, as the financial management is rarely subjected to close scrutiny and intelligent criticism. Mr. Kuichling.

It may also happen, in the second case mentioned, that after some years the industries, the establishment of which in the community caused a rapid growth of population and consumption, are moved to another town. Such an occurrence is generally followed by the abrupt departure of many inhabitants, and years may pass before other industries and people are attracted to the place. In the meantime the water-works plant is too large, and its revenue is greatly diminished, but the interest and depreciation accounts remain the same, and the enterprise must struggle along with another series of annual deficits unless the rates are readjusted.

These possibilities of additional investment of capital followed by unexpected reduction of revenue, constitute the so-called commercial risk involved in all private enterprises for the service of the public. Experience has amply demonstrated that this risk is as real as that of loss by fire, and adequate compensation for taking it is necessary in one form or another. In ordinary commerce the compensation is in the form of a tangible annual profit, or excess of revenue over all expenses for interest, depreciation, operation, and taxes; but, when the water-works do not earn such a profit, the risk remains uncompensated, and the security of the investment is impaired until the rates are raised. The method of making compensation for this commercial risk during the period of insufficient income seems to have received little attention, and in valuations the subject is usually ignored. It can be treated, however, as a yearly expense, like fire insurance or the interest charge on borrowed capital, and be measured by the difference of from 1 to 3% between the high rate of interest required when the security is doubtful and the low rate granted when the security is unquestioned. The sum of these excess interest charges on the capital invested, or the corresponding depreciation of the bonds or debentures of the company during this period, measures the risk and becomes a part of the cost of creating an ultimately remunerative income.

The question now arises as to what is meant quantitatively by a remunerative income or "fair return." This is commonly estimated as a certain percentage of the capital actually invested in the plant, and must be sufficient to cover all charges for interest, depreciation, operation, and taxes, together with a reasonable excess for profit, which will obviously depend on the risk involved in the permanency of the income. Under the most favorable conditions, a profit of 2% can be regarded as a minimum, as this will merely suffice to establish a reserve fund to meet a variety of possible expenditures for keeping the original capital intact. For less favorable conditions, the profit must

Mr Kuichling.

be correspondingly greater, and in cases where the risk of the permanency of the enterprise is large, a profit of from 6 to 10% is not unreasonable. It may not be possible to frame a general rule for determining a reasonable profit, but the citation of a number of instances where it has been fixed by judicial proceedings would be of great value.

In financial circles the matter of profit is dealt with rather abruptly. Money will not be loaned readily for making extensive improvements to a water-works property unless the company can show a revenue of more than 8% on the invested capital, after paying operating expenses and taxes; and, in some instances, it was stated that a loan would not be granted unless such revenue exceeded 12 per cent. The magnitude of the desired loan is necessarily an important element in the case, as well as the probable growth of population in the city; but the figures mentioned indicate that the normal interest and depreciation charges are taken together at about 7% on the investment prior to seeking the loan, and that a profit of from 1 to 5% on the capital is essential to the continued solvency of the concern. These figures, however, are not to be regarded as the measure of "fair return," but only as the lower limit where substantial profit begins.

An important element in the computation of the going value of a water-works property is the length of time required for the comparative plant to obtain the same number of consumers as the real plant has at the date of the valuation. The company usually acquires its business slowly, because the introduction of water into a building not only involves a considerable initial outlay for the various fittings, but also entails the provision of a drainage or sewerage system. The latter is generally of small extent in towns where public water-works do not exist, and hence a large expenditure by the great majority of the inhabitants becomes necessary before the normal capacity of the company's plant can be developed. While many of the inhabitants may be inclined to take the water at the outset, yet they realize that they cannot afford the additional cost of the public sewerage work, and, consequently, they postpone their applications for water connections. For this reason, many years elapse before the company succeeds in obtaining 80% of the population as regular consumers.

The same is likewise true of municipal water-works, in spite of the powerful incentive to every householder to make early use of the public water supply in order to avoid paying for deficiencies of revenue through increased general taxation. Thus, in the case of a certain city in the State of New York, in which the water-works were completed in 1876, when the population was 83 250, and 3 260 service connections had been made during construction, it was found that in the course of the succeeding 14 years the number of such connections

increased at a nearly uniform rate of 1 540 per year, the total number in 1890 being 24 864, while the population had grown to 133 896. Similar figures are also found for several cities in other States during a period of from 10 to 15 years following the completion of the water-works, although, in some cases, the average number of service connections made within the first 3 or 4 years was somewhat greater than the nearly uniform average for the succeeding years. Mr. Kuichling.

On the other hand, the authors allow only a very few years for the comparative plant to acquire all the connections and business of the existing plant, on the ground that the work should be done in the shortest possible space of time. This tends to diminish the going value considerably, as it reduces to a minimum the period during which the comparative plant will have an inadequate revenue. It must also be remembered that there is a practical limit to the speed at which house connections can be made in a city, as a large number of streets in a district cannot be rendered impassable by transverse pipe trenches at the same time, without dangerously increasing the fire hazard and seriously interfering with mercantile operations. Therefore, it seems that a somewhat longer period should be taken for the comparative plant to make these connections and acquire the business of the water company. A good understanding of this feature would be greatly facilitated by the presentation of data relating to rapid work of this character.

A few remarks are also ventured on the subject of the cost of reproducing the plant. The Courts have generally agreed on the principle that this should be based on the prices of material and labor at the time of the valuation, instead of taking the actual cost, as shown by the company's records. The reasons given for this ruling are, that in most cases properly itemized records of the original cost of the plant are not available, and, furthermore, that inasmuch as the prices of materials and labor are constantly fluctuating, no injustice will be done on the average to either party by taking the present prevailing prices. It is also asserted that, if the purchase is made during a term of high prices, the buyer must assume the responsibility for paying more than the actual cost, and, conversely, if the transfer is made when prices are low, the owner of the property should bear the loss. This principle does not seem to be equitable. If trustworthy figures of actual cost are not available, the prices of materials and labor at the time of construction can almost always be ascertained, and the estimate made in accordance therewith. The results will usually be much nearer the true cost than any estimate based on present prices.

H. F. Dunham, M. Am. Soc. C. E. (by letter).—In the discussion of a paper on "The Valuation of Water-Works Property,"* by Wynkoop Mr. Dunham.

* *Transactions*, Am. Soc. C. E., Vol. XXXVIII, p. 115.

Mr. Dunham. Kiersted, M. Am. Soc. C. E., the writer referred to two features, incidental to valuations, namely:

(1) The use of curves in fixing going values. This may not have been the first reference to such use, but curves have been utilized to good purpose since that date.

(2) A casual mention of the importance of prosperity in going-value appraisals and of "conditions not well understood by any one."

Rapid progress has been made, but there is still an unsifted residue, variable in magnitude as the years pass, hard to realize, difficult to formulate, but of greater import than interest allowances during construction and many of the refinements about which one likes to be so exact. How well does the "comparative plant method" of determining going value make just provision for some of these more or less concealed but common conditions? To what extent should a valuation problem be dependent on the solution of other valuation problems? Is it safer to generalize, in reaching a going value, than it is in reaching a good design for a particular place and works?

One or two illustrations may be helpful: A contract under a franchise implies good faith and fairness on the part of both contracting parties. Suppose a water company fails to carry out the spirit of its contract, builds feebly, manages badly, and serves the city poorly. In a valuation these facts are considered, and the city is not obliged to face an award as large as it would have been under a generous treatment of the contract by the water company. In other words, while both parties suffer, the violator of the terms of the contract suffers more. The "comparative method" may need but slight modifications to sustain a reasonable going value.

In another case, at the very beginning the city insists on a given location for the works (perhaps under the advice of a prominent engineer), and a contract is made. Time shows the location to be bad, and the water company is obliged to build again, at unexpected cost, and to abandon its first construction. Both parties have acted fairly. Will the "comparative method" yield for this case a fair going value, or take into account the fact that had the city decided to build with its own, instead of an agent's capital, it would have been obliged to suffer the entire loss in the change of base?

Let it be supposed that a city fails to pay its rentals as contracted, with the result that the company is impoverished, unable to make needed extensions, and goes to a valuation with a dwarfed plant and a big bill for litigation. How well will the "comparative method" fit this case?

Franchise contracts usually provide that the city shall exercise police power and protect the company's property from harm and the source of supply from pollution. Suppose a city fails in these important particulars and, through some technicality, cannot be made to act; in many instances the city with its own sewage is the active

transgressor. There follows lasting injury to the property. Can the "comparative method" correct this or prevent the city from securing a works at a low award as the direct result of its own failure to comply with the terms of the contract? In this instance should the value, including the going value, be determined at the date of taking the real plant, without reference to its value at an "imagined" early date when contract wrongs shall have been at least partly righted? Mr. Dunham

Exact legal terms may not be readily introduced, but a general view with rather broad judicial power is not inconsistent with the work of an appraiser. If he can pass on the value of a net revenue, he should, when required, be able to pass on the circumstances under which any revenue is created, maintained, or predicated. He may not find a curve or algebraic expression covering circumstance and going value, but he should not be unmindful of either.

The appraiser, in his efforts to secure a going value as a part of a just award, should not be restricted by compilations from other cities, by new methods, new formulas, or even new rules from the Courts. These are all derived from cases unlike his own. They are valuable aids, but should not be substitutes for vital individual work, no less searching than that which has been embodied in the aforesaid new and valuable methods and rules.

Charles B. Buerger, Assoc. M. Am. Soc. C. E. (by letter).—In this paper going value is defined as the value of a created income, and the writer wishes to call attention to the importance of the amount of this income, relative to the physical value of the plant, as affecting the going value. Mr. Buerger

To the investor in the securities of the water-works, or to the tax assessor, the capitalization of this income, as effected by the method elaborated in this paper will no doubt be satisfactory, and the value thus derived will seem to them a fair one, but a not uncommon mover for the valuation of a water-works property will be the community served by the water company. The purpose will then be the determination of a fair purchase price, and particularly a determination of the fairness of the rates charged by the company. The valuation will be made either for the Courts or for arbitrators, and, on this valuation, the Courts or the arbitrators will wish to allow a reasonable return and to fix rates accordingly; but if this value is obtained by crediting the water company with the capitalized income resulting from the excessive rates in force, and then basing the rates on the excessive going value resulting, added to the physical value, the figures thus obtained may fail to satisfy the people who are to pay these rates.

Take an imaginary case of a water-works which has for years, or even since its construction, enjoyed a gross income 50% in excess of that obtained by other water companies under similar conditions for the same service, and that the Courts or a Public Service Commission asked Messrs. Alvord and Metcalf to give the value of the plant,

Mr. Buerger.

including "going concern value." They would hardly capitalize this excessive income and report it to the Court for rate-making; and yet their paper leaves no alternative method open.

To put this case in concrete form, assume that the plant, as given by Table 1, remains the same with the exception of an increase in the gross income of 50% obtained by an increased rate without change in operating expenses, taxes, and other expenses. Assume, again, Table 2 and Table 3 without change for the comparative plant. Table 4, by changing Column 2 by increasing the gross income, would appear as in Table 8.

TABLE 8.—Existing Plant; Annual Income and Expense.

Year ending Dec. 31st. (1)	Gross income. (2)	Operation, etc. (3)	Net income. (4)	Depreciation. (5)	Net income for fixed charges and dividends. (6)
1910.....	$1 575 000	$360 000	$1 215 000	$90 000	$1 125 000
1911.....	1 620 000	370 000	1 250 000	91 000	1 159 000
1912.....	1 665 000	380 000	1 285 000	92 000	1 193 000
1913.....	1 710 000	388 000	1 322 000	93 000	1 229 000
1914.....	1 742 000	395 000	1 343 000	94 000	1 249 000
1915.....	1 785 000	402 000	1 383 000	95 000	1 288 000
1916.....	1 819 000	409 000	1 410 000	96 000	1 314 000
1917.....	1 852 000	414 000	1 438 000	97 000	1 341 000
1918.....	1 892 000	420 000	1 472 000	98 000	1 374 000
1919.....	1 910 000	425 000	1 485 000	99 000	1 386 000
1920.....	1 944 000	430 000	1 514 000	100 000	1 414 000

The figures for depreciation are the same as in the authors' Table 4, although they might properly be increased a little.

Table 9 is obtained on the same basis.

TABLE 9.—Comparative Plant; Annual Income and Expense.

(From operation, exclusive of interest on unemployed capital.)

Year ending Dec. 31st. (1)	Gross income. (2)	Operation, maintenance, taxes. (3)	Depreciation.		Net income applicable to fixed charges and dividends (6)
			Base. (4)	Amount at 1 per cent. (5)	
1910.....	.....	.....	.....	.....	.....
1911.....	$75 000	$118 000	.....	.....	$43 000
1912.....	292 000	195 000	.....	.....	97 000
1913.....	577 000	255 000	$2 123 000	$21 000	301 000
1914.....	862 000	305 000	3 106 000	31 000	526 000
1915.....	1 140 000	344 000	6 065 000	61 000	735 000
1916.....	1 384 000	372 000	7 152 000	71 000	941 000
1917.....	1 594 000	394 000	9 082 000	91 000	1 109 000
1918.....	1 762 000	410 000	9 082 000	98 000	1 254 000
1919.....	1 875 000	422 000	9 082 000	99 000	1 354 000
1920.....	1 944 000	430 000	9 082 000	100 000	1 414 000

Tables 10 and 11 are carried through on the same basis as Tables 6 and 7.

Mr. Buerger.

TABLE 10.—First Trial Computation of Net Annual Income (from Operation and from Interest on Unemployed Capital) of Comparative Plant.

Assumed Capital in hands of Investor-Builder on January 1st, 1910:

Value of physical plant	$8 600 000
Interest during construction	482 000
Assumed going value	3 818 000
	$12 900 000

Year ending Dec. 31st. (1)	Net income. (2)	Physical plant and interest. (3)	Going value. (4)	Total amount invested. (5)	Amount earning 4 per cent. (6)	Years expenditure. (7)	Interest, Column 6, at 4 per cent. (8)	Interest, Column 7, at 1 per cent. (9)	Total interest. (10)	Annual income. (11)
1910	0	$824 000	0	$824 000	$12 076 000	$824 000	$483 000	$8 000	$491 000	$491 000
1911	$43 000	2 623 000	$598 000	3 221 000	9 679 000	2 397 000	387 000	24 000	411 000	368 000
1912	97 000	4 806 000	1 302 000	6 108 000	6 792 000	2 887 000	272 000	29 000	301 000	398 000
1913	301 000	6 865 000	1 969 000	8 834 000	4 066 000	2 726 000	163 000	27 000	190 000	491 000
1914	526 000	8 252 000	2 553 000	10 805 000	2 095 000	1 971 000	84 000	20 000	104 000	630 000
1915	735 000	9 082 000	3 016 000	12 098 000	802 000	1 293 000	32 000	13 000	45 000	780 000
1916	941 000	9 082 000	3 374 000	12 456 000	444 000	358 000	17 000	4 000	21 000	962 000
1917	1 109 000	9 082 000	3 608 000	12 690 000	210 000	234 000	8 000	2 000	10 000	1 119 000
1918	1 254 000	9 082 000	3 747 000	12 829 000	71 000	139 000	3 000	1 000	4 000	1 258 000
1919	1 354 000	9 082 000	3 816 000	12 898 000	2 000	69 000	0	1 000	1 000	1 354 000
1920	1 414 000	9 082 000	3 834 000	12 916 000	—16 000	18 000	—1 000	0	—1 000	1 413 000

Mr. Buerger.

TABLE 11.—TRIAL COMPUTATION OF GOING VALUE AS OF JANUARY 1ST, 1910.

Year ending Dec. 31st.	Total Net Return.			Present Worth Excess Earnings.	
	Existing plant.	Comparative plant.	Excess.	Amounts.	Cumulative amounts.
(1)	(2)	(3)	(4)	(5)	(6)
1910	$1 125 000	$491 000	$634 000	$598 000	$598 000
1911	1 159 000	368 000	791 000	704 000	1 302 000
1912	1 193 000	398 000	795 000	667 000	1 969 000
1913	1 229 000	491 000	738 000	584 000	2 553 000
1914	1 249 000	630 000	619 000	463 000	3 016 000
1915	1 288 000	780 000	508 000	358 000	3 374 000
1916	1 314 000	962 000	352 000	234 000	3 608 000
1917	1 341 000	1 119 000	222 000	139 000	3 747 000
1918	1 374 000	1 258 000	116 000	69 000	3 816 000
1919	1 386 000	1 354 000	32 000	18 000	3 834 000
1920	1 414 000	1 413 000	1 000	0	3 834 000

The total effect is the increase of the going value from $1 880 000 to $3 818 000, and an increase of the total value of the plant from $10 962 000 to $12 900 000. The going value is doubled, and the plant value is increased 18 per cent.

It would seem possible, in a case like this, to make new assumptions for the gross income for both the existing and the comparative plants at lower rates, repeating all the figures until a total value is obtained on which the basic net income is a fair return at a predetermined fair ratio.

Two points in the authors' interest calculations seem open to criticism. It is assumed that all the capital for the construction of the comparative plant is to be available at the beginning of operation, that is, the date of the valuation. As the actual construction must by assumption take a period of 6 years, it could reasonably be taken that some of this capital is obtained at a later time.

Also, going value, representing the present worth of the excess income of the existing plant over that of the comparative plant for the 10-year period, is obtained on a 6% basis (Table 7). In Table 6, going value capital is taken as invested so as to draw 4 per cent. More correctly, Column 6, of Table 6, should be divided into two parts, one going value capital earning 6%, and the other temporarily invested capital drawing 4 per cent.

Mr. Gillette.

HALBERT P. GILLETTE, M. AM. SOC. C. E. (by letter).—At the very outset of his work the appraiser of a public utility must adopt one of two premises:

First.—That a public service corporation is entitled to a fair return on its investment and for its managerial services throughout the entire period of those services.

Second.—That a public service corporation is entitled only to what it can get under more or less competitive conditions throughout the entire period of its service to the public. Mr. Gillette.

From each of these premises logically follow certain principles to which the appraiser must adhere.

Among appraisers, nearly all disagreements as to theories or principles arise because of differences as to these two premises. It is important, therefore, that an appraiser shall state which premise he has decided to select. Unfortunately, there has been so little published discussion of the principles and methods used in appraising public works, that comparatively few appraisers have had any clearly defined ideas of the fundamental principles involved. The writer welcomes this opportunity to outline some of his own conclusions, more particularly those which follow as logical corollaries of the first premise stated.

The authors have adopted the second premise in their appraisals of water-works, for they specifically exclude any investigation of the actual deficits incurred by a given water company in developing its business.

They also exclude any investigation of the actual history of the construction of the given water-works, ignoring all difficulties actually encountered and expenses incurred, unless those difficulties and expenses would be encountered at present. In brief, their theory of appraisal involves consideration only of the cost of producing a duplicate plant and business under present conditions. It is easily seen that this involves the premise that competitive conditions control their ideas of valuation. Moreover, they make this position very plain when they assume that the public, like the capitalist, stands "at the parting of the ways," trying to decide whether it shall buy the existing water-works or duplicate the plant. This is the competitive system of barter in a nutshell.

In America many water-works plants have been purchased by municipalities at the expiration of franchises. It so happens that public opinion has usually been inflamed against corporations, because public ignorance has led to the belief that enormous profits have been reaped by public service corporations. Hence, whenever the public has had a chance to secure a water-works plant at the expiration of a franchise, it has used the same methods in bargaining that its individual members use in their every-day competitive strife. It does not follow, however, that this essentially unjust attitude toward public service corporations will continue indefinitely; and, if it does not continue, every appraisal of a public utility must be based on the first premise mentioned, and not on the second.

Already, in the case of railways, it is found that most Railway Commissions are endeavoring to adhere to this first premise. Where

Mr. Gillette.

they do not, they are usually reversed by the Courts. It may now be said to be an established principle of law, that a public service corporation is entitled to a fair return on its investment and for its service as a public agent; therefore, in the case of corporations whose franchises have not expired, it becomes legally impossible for an appraiser to adopt any other premise. As already stated, if a franchise has expired, the question of fair return on the investment may be ignored by an unjust public, and competitive conditions may determine what it will pay for a water-works or other utility.

Threats of building a competing system of water-works may be, and generally are, used to force down the price which a company is morally entitled to receive. In such cases one passes from the realm of engineering to a sort of mart, where the engineer usually does not care to enter.

The writer conceives that the authors' "comparative method" is really not one which involves an appraisal of the cost of reproducing an existing water-works, but that it involves producing a hypothetical water-works, similar to the existing one, and, if carried to its logical conclusion, should involve designing an entirely new plant, for the question is then not primarily one of physical entities possessed by the existing plant, but of the service which it renders. In that event, the appraiser should not first ask whether the existing pipe lines are of wood, steel or cast iron, nor should he first consider existing diameters of pipe, size and kind of pumping engines, etc., but he should design an entirely new plant to perform the service required. Having designed such a plant, he should estimate the value of the existing plant by comparison with the new one.

This should be done in the following way: To the first cost of the new hypothetical plant add the capitalized cost of the estimated average annual operating expense, and call this sum the total capitalized cost of the hypothetical plant. From this total capitalized cost, deduct the capitalized annual operating expense of the old plant, and the remainder will be the appraised value of the latter, not including going value. In both cases the estimated annual operating expense should include amortization.

There is no escape from adopting this method, if the second premise is accepted. As a matter of fact, some appraisers have attempted roughly to carry out this plan, although, as far as the writer knows, this has never been followed to its logical conclusion in every detail, as above outlined, in any given case.

Let us consider a few examples of palpable injustice to public service corporations which would occur if all appraisals were based on the second premise. Practically all public service corporations have grown from small enterprises to their present condition; hence they have constructed their plants piecemeal. Piecemeal construction, as

Mr. Gillette.

one of the authors has himself proved elsewhere with admirable thoroughness, is more expensive than wholesale construction—often very much more so. Not only is the unit cost of labor higher on small jobs, but freight rates on less than car-load lots, and prices of materials on small orders, are higher. Therefore, even though the daily rate of wages and the base price of materials remain stationary, the unit costs of construction on a plant built piecemeal are necessarily much higher than on one built all at one time. If engineers adopt the second premise, therefore, they will assign values to the existing property of public service corporations far below the cost of production, for the second premise prevents consideration of the actual conditions of original construction.

A street railway exists in a city where heavy cuts and fills were made by the railway in building its lines into the suburbs. Subsequently, large areas of the city were re-graded, great hills being cut down and valleys filled. This street railway paid vast sums for its original grading and for the re-grading. Yet, if an appraiser should look at the problem of appraisal as one of ascertaining the "cost of reproduction under present conditions" only, he would exclude all this grading cost, and would become the instrument by which nothing short of highway robbery would be perpetrated on the street railway corporation.

To cite another example from the writer's recent experience in appraisal work, let us consider what allowance an appraiser should make for clearing and grubbing a reservoir site and the right of way for a flume and pipe-line. To-day trees are still standing along the right of way, but a few years hence they may be entirely gone, and nothing may remain to show the original cost of clearing. Because the trees are gone, shall the appraiser omit this item of actual cost of producing the plant? If he does, he will not reproduce that plant at all in his estimate, but will produce some hypothetical plant. Many specific instances occur to the writer illustrative of the general proposition that no appraisal is likely to be fair to a public service corporation unless the appraiser becomes familiar with the entire financial and physical history of the corporation property. Contention may be made, of course, that every corporation should have made enough profit to reimburse itself for all expenditures upon which it cannot expect repayment from the public when the day shall come that the public shall have the corporation by the throat. What would naturally follow if every corporation raised its rates enough to provide sinking funds to enable it to recoup itself for every such expenditure? Without question, the Courts would then be forced to maintain the justice of such an increase in rates or lay down the rule that the public should not be burdened with the higher rates, but should be prepared to pay the actual cost of the plant and of its

Mr. Gillette.

development expense, in the event of its ultimate purchase. In fact, the writer confidently expects this latter proposition to become the universal law.

Adopting, as the writer does, the first premise, namely, that a public service corporation is entitled to a fair return on its investment and for its managerial services, most of the perplexities confronting an appraiser vanish. It follows logically that the appraiser must base his appraisal on the actual conditions under which the property was built and operated. If trees were cleared, then he must allow for the cost of clearing, although not a tree may now be standing. If streets were graded, then that grading must be estimated, though to-day the entire city is as level as a floor. If quicksand was encountered in laying a pipe-line, then the added cost of excavating it must be allowed, even though subsequent works have drained the line so that it no longer has a yard of quicksand. If money was spent to educate the public to the use of the commodity sold by the corporation, then that money is a development expense which must be allowed, even though the expense would not now be incurred by a new corporation of like character. If the corporation has built railway lines to develop a country, and has not only spent money to get people to settle there, but has experienced deficits below a fair return on its investment until the country has become sufficiently populated, then this development expense must be allowed. In brief, the entire history of the public utility must come within the appraiser's knowledge and consideration.

The writer has elsewhere given reasons for preferring the use of the term, "development expense," to "going value." The latter conveys entirely different meanings to different men, and it is not likely that any unanimity will ever be reached. The writer has defined development expense as being the cumulative deficits below a "fair return" up to the point when a "fair return" is thereafter obtained. It is the sequel to interest during construction. In fact, it is perfectly logical not to consider interest during construction as a separate item of expense, but to regard the development period as starting on the very day the first money is spent for organization or construction. The deficit of each year should be added to the construction cost of that year, and this sum should be added to the total plant account at the end of the previous year.

In the discussion of Mr. Riggs' paper on "The Valuation of Public Service Corporation Property,"* the writer has already outlined at some length the reasons for development expense, and the methods by which it is determined.

The writer has noted with great interest the recent decisions of the Wisconsin Public Service Commission, whereby, as he infers, it has

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 275.

Mr. Gillette.

practically been decided to adopt the method of determining development expense from the deficits deduced from the accounting records of public service corporations. This is a most important step in the march of progress toward a rational and just solution of appraisal problems by public service commissions. The authors fail to concur in this conclusion because they have adopted a theory which rests on the assumption that competitive conditions shall determine the value of public utilities. The writer cannot accept such a theory, first, because it violates the principles of pure justice to public service corporations; and second, because it introduces a competitive condition at the time of appraisal, although a monopolistic condition has existed up to that time, and yet one in which the monopolist has not had entire freedom of action. The monopolist—the water company in the case under discussion—has usually been limited as to rate charges, and held to many other limiting conditions defined in its contract with the city, namely, the franchise. It has not then been free, but has been an agent under restraint. As such an agent, therefore, it owes the public a service which, at times, must be given even at a temporary loss, such as when new pipe-lines are extended into sparsely settled districts. In return, however, the public owes to this same agent far more than it would owe to an itinerant peddler on its streets, for it owes this agent a fair compensation for its services from the beginning to the end of its franchise, and, at the end, it owes such deficiency in the compensation as is represented in the money invested in the existing plant and in developing the business possessed by that plant.

The writer believes that this is the view which most appraising engineers would gladly take, were they entirely free from the effect of that force called "public opinion." In the water-works field, "public opinion" has been abnormally strong, and has led to several Court decisions which, eventually, will be regarded as unfair. In the railway field, public opinion has not yet affected the higher Courts to any degree, and there has been a greater measure of justice there in consequence. The celebrated decision of the Supreme Court in the Nebraska railway case of Smythe *vs.* Ames still remains the standard by which the judgment of the most enlightened of the State railway commissions is guided. In that case, the Court held—and the principles are equally applicable to water-works—that a rate-making commission must ascertain the actual cost of construction, the cost of reproduction new, the present value, the amount and value of stocks and bonds outstanding, and several other specified factors; and it was careful to add that still other factors might properly be needed to guide the rate-regulating body.

It is evident that the Courts themselves are not as yet wholly clear on the theories of appraisals, to say nothing of the more complex

Mr. Gillette. theories of rate-making. The writer believes, therefore, that one of the important functions of appraising engineers is to enlighten the Courts on the facts and theories. He agrees heartily with the authors that it is distinctly the engineer's function to determine the going value or development expense. He concedes that the authors have worked out an admirable solution of the problem, based on their fundamental theory of appraisal, but he contends that the theory is wrong in principle, and can never prevail long in a community which follows the golden rule in its dealings with its agents, the public service corporations.

Mr. Fuller. GEORGE W. FULLER, M. AM. SOC. C. E. (by letter).—This paper is an important contribution to the literature of a subject on which widely different opinions still seem to prevail. As applied to water-works valuations, the element of going value has doubtless been worked out, as stated in the paper, to a more logical conclusion than in the instances of plants of many other public utilities, particularly those in which the element of competition is a substantial factor.

Going value, in the writer's opinion, is clearly an element of value of material proportions in the case of a water-works plant with an established profitable business. The estimated cost of reproducing the established business, of a volume corresponding with the date of valuation, is the best available plan of procedure. It is necessary to proceed, however, in a manner which on the one hand will give due recognition to the owner of the increased worth of a plant with an established business, as compared with a similar plant without such business, and, on the other hand, will not include in going value any items which are taken care of already under other headings.

The writer agrees with the authors as to the recent suggestion of the Wisconsin Public Service Commission to use for going value the early expenses of a company in building up its income to the point where it is sufficient to carry depreciation and fixed charges as well as the operation and maintenance expenses. For general application, this seems to be an error, in that it deals with a volume of business which may be widely different from that of the property actually found at the time of valuation. Perhaps the most illogical feature of this suggestion is the tendency which it has to give a high going value to a property where early losses continued for a long period, and correspondingly a low going value to a plant with remunerative business and a short term of actual deficiency in earning capacity to meet all reasonable charges.

This paper is prepared on the general theory of reproducing the actual income of an existing plant at the date of appraisement. Where rates are stipulated by contract, or are subject to the control of the company without review by Courts, that procedure is logical and

proper in the case of an appraisement for purposes of transfer or sale; but when it comes to establishing the going value of a water-works plant for purposes of rate-making, or where the fairness of rates is disputed, it is necessary to take other factors into account, because the element of going value is in itself a factor to be included as a portion of the investment upon which a reasonable return should be earned. Mr. Fuller.

It is feasible, of course, to proceed with actual earnings and then to appreciate or depreciate the going value for purposes of rate-making, according to the return obtained from a given schedule of rates for the service and the value of property actually found. The paper would perhaps be more useful to those interested in rate-making if these features of adjustment outlined on pages 338, 339, and 353, under "Stage of Development," were to be elaborated somewhat.

Losses of interest during the period of construction of a comparative plant, incidental to the investor handling his funds so as to pay for construction work from month to month, should be charged to capital account rather than to going value, according to the writer's custom hitherto. It is not plain to him why these losses are incidental to building up the business rather than to construction cost, as the latter has to be met prior to the commencement of operations.

When it comes to including in going value the difference for a term of years between a 4% and a 6% rate of interest, this item becomes a substantial one. It seems to the writer that this procedure is somewhat debatable, particularly in the case of a plant requiring 10 years for construction and where a return of only 4% on unemployed capital is allowed. Is it not reasonable to assume that, for a term of years, an investor who is willing to go into the water-works business could find readily marketable securities of an industrial character which would yield a greater rate than 4%, and which would be as stable as the ordinary water-works bonds of a well-managed water company?

Another query is this: Should the going value include an allowance to the existing plant for profits on operation, above, say, 6%, during the period of construction of the comparative plant? The writer would be interested to learn the views of the authors for a case where the comparative plant was completed on the day that the franchise of the old plant expired, and also when the completion was one year before or one year after the franchise expired.

Another feature which is illustrated in this paper is the transfer of the interest account upon a certain portion of the plant, when it is completed and put in service, from the "loss of interest during construction" (capital account) to the operating account (as fixed charges). This seems to be inconsistent with several statements in the paper, notably the next to the last paragraph where, as the writer

Mr. Fuller. agrees, "the [interest on] fixed charges should not be so included" as operating expense. It seems to the writer that this may complicate rather than aid a clear understanding of the subject. Would it not be possible to arrive at the same result by keeping construction accounts and operating accounts clearly separated during the entire construction period of the comparative plant, and then, by proper methods of interest and discount, arrive at the same result without this confusion of interest charged partly to operating expense and partly to "loss of interest during construction," or capital account?

A factor influencing largely the amount of going value is the rate of acquisition of income by the comparative plant from the time of completion of its construction, either in its entirety or perhaps as to a portion thereof, until the income or volume of business has reached the level of that of the existing plant. The authors cite several theories or views which are frequently met in work of this character. This important item (pages 336 and 337) is left in rather indefinite shape, and without a comprehensive review of the influence of various local conditions. For instance, it makes a good deal of difference as to the rate of acquisition of business by the comparative plant whether it is so located that both domestic and industrial consumers can quite readily provide themselves with water under fairly convenient conditions, or whether other means of providing a water supply to the individual householder or business establishment are practically out of the question. The latter condition is not entirely a fanciful one, as in the arid regions the individual is frequently precluded by expense from securing a satisfactory individual water supply, and the same is true in some of the coal regions where in coal mining operations the underground water has been removed and where surface waters are highly polluted with acid wastes. On the other hand, where water-bearing strata fairly near the surface are protected from pollution by a practically continuous stratum of clay, the element of competition of the comparative water-works plant supplying pressure water is much different. A discussion by the authors of the significance of certain types of local conditions would probably add to the usefulness of this important and highly creditable paper.

Messrs. Metcalf and Alvord. Leonard Metcalf and John W. Alvord, Members, Am. Soc. C. E. (by letter).—The writers have been much gratified by the apparent interest evoked by this paper and the valuable discussions which have been contributed to it. It has interested them, in the light of the care exercised to avoid ambiguity, to see how the reader's interpretation of an author's statement may differ from an author's real view.

The writers recognize clearly and admit frankly that no single method of valuation can determine value in all cases. There are always many modifying conditions and local circumstances which

have an important bearing on the valuation of any plant. Breadth of engineering experience, due appreciation of the commercial and financial sides of the problem, coupled with a desire to see full justice done to both parties, are the only safeguards to the judicial analysis of any problem in valuation. It must also be recognized that the determination of going value, or development expense, involves the determination of but one element of the total value, and, while the writers believe that the going value of a plant is always a positive one, they are of the opinion that there are many cases where the commercial, or true, value of the property may not be as great as the several elements of value entering into the property, separately appraised. No value is complete until the final summing up has been made and the value as a whole has been judicially weighed and gauged by the several available standards of length which the particular problem under investigation may seem to make applicable. Messrs. Metcalf and Alvord.

As far as the comparative method of determining going value is concerned, the writers do not contend that this method of analysis is infallible; indeed, they have seen it badly abused in its application by engineers playing the role of advocate rather than disinterested expert or judge. Nevertheless, they have found the method of analysis helpful as an aid in forming judgment, and have seen many cases where its application in the hands of fair-minded men, called in by opposing interests, has led to very similar results—results which apparently satisfied the judgment of counsel and of the tribunal passing on the case, as well as that of the experts presenting them.

While the casual reader would be likely to infer from Mr. Gillette's discussion that his results would probably be very divergent from those obtained by the comparative method, the writers apprehend that in the final analysis the results might not be very different. One of the chief difficulties with which appraisers usually meet (particularly in evaluations by the Court) is that no continuous record of past operations is available. Within the experience of the writers, this has almost always been the case. Under such circumstances, obviously, Mr. Gillette's method, or that of the Wisconsin Railroad Commission, is impossible of application; but even in those cases where full records are available, the analysis of the past records (with all the errors in methods of accounting, which are likely to creep into the management of properties of this sort) is no easy matter, and may involve quite as great uncertainties as the determination of the fair reproduction cost of the property.

The writers cannot accept Mr. Gillette's statement of their position as being either fair or sound, though they are of the opinion that the results to be obtained by either of these methods would not be very far apart.

Messrs. Metcalf and Alvord.

There is no doubt that the introduction of public service commissions, or Government control of quasi-municipal corporations, will lead to radical changes in methods of operation, as well as methods of valuation, and while it is, of course, desirable that engineers should do all that they can to outline sound principles of valuation, it is not possible for them to ignore the existing laws relating to that subject, as interpreted by counsel and Court, and certainly no engineer would be justified in jeopardizing his client's interests by ignoring any existing limitations of this sort. The writers believe that we are approaching a condition such as that which Mr. Gillette describes, but there can be no doubt that the Courts have laid down the rule that original cost is not of controlling importance, and any engineer who bases his judgment of value of such properties solely on such an analysis or criterion may give his clients advice which may cause them serious financial loss. Mr. Gillette says that he

> "conceives that the authors' 'comparative method' is really not one which involves an appraisal of the cost of reproducing an existing water-works, but that it involves producing a hypothetical water-works, similar to the existing one, and, if carried to its logical conclusion, should involve designing an entirely new plant, for the question is then not primarily one of physical entities possessed by the existing plant, but of the service which it renders."

In this he is wholly wrong, if he is seeking to interpret the actual views of the writers, for the Courts have clearly laid down the rule that the existing property shall be valued, and not an equally efficient plant. In his statement: "if engineers adopt the second premise, therefore, they will assign values to the existing property of public service corporations, far below the cost of production, for the second premise prevents consideration of the actual conditions of original construction," he is also wrong. Such has not been the experience of the writers, even as judged by comparative estimates made by other engineers on the same or the opposite side of the case at issue. One might almost question, in the light of this statement, Mr. Gillette's understanding, or interpretation, of the meaning of the phrase "reproduction cost."

The writers agree heartily with Mr. Gillette's statement as to the bearing which the history and experience of the works in question should have on the estimates of cost of its production or reproduction. Why should not the same consideration be given to original conditions under the reproduction cost theory as under the original production cost theory? Without going into detail, it is perhaps sufficient to state that, in a recent joint valuation by the authors, the very elements which Mr. Gillette says should be included in valuation, and could not be included under the reproduction cost theory, were included by the writers, as, for instance, the removal of heavy earth embankments, the

existence of which cannot be wholly traced to-day; the removal of trees; and the removal of houses and buildings. The writers conceive that the past history of the works and of their construction is of the utmost importance to the appraiser, if he would render fair judgment on the value of the property. Messrs. Metcalf and Alvord.

As long, however, as the Courts shall adhere to their past view, that original cost is but one criterion of value and that reproduction cost is another, there would seem to be as great justification for considering the reproduction cost of the going value as the reproduction cost of the physical plant. Whether the going value be treated as development expense or going value, matters not, provided consideration be given to the cost of these factors in the up-building or operation of any public service corporation.

Incidentally, it is perhaps worthy of attention that the analysis by the Wisconsin Railroad Commission—fair as it has been in most respects, and aiming to deal with the cost of production rather than the reproduction theory as far as going value is concerned—has dealt with what to-day would constitute a reasonable profit, rather than with the actual prevailing rates of interest on projects of like hazard in a series of years which has passed. Its analysis, on a basis of average rate of from 6 to 7%, rather than on a single rate basis, has, of course, tended to modify this result; but an examination of the actual rates of return on capital during the past 30 years would show a considerable variation in such rates; therefore, their analysis has not been based on actual past experience, in all cases, but on the Commission's view as to what may constitute a reasonable assumption as to rate of return on the property of the public service corporation. Mr. Kuichling's discussion along this line seems to the writers sound, though they recognize the difficulty of drawing the line between cumulative losses in the early development period, or reconstruction periods, of the corporation's life history—which should be made good by later excess earnings of the corporation—and a losing contract, which, under the present interpretation of the Courts (and in Mr. Gillette's Utopian theory, however desirable the latter may be), would clearly have to fall, in some measure, on the owners of the property. In such cases the appraiser will be likely to err in an over-liberal attitude toward the corporation, if he is fair-minded, and this is desirable, for, unless capital is fairly protected in the work which it does for the public, the public must ultimately pay a higher rate because of the unfair treatment which it has given to its public servant.

The determination of the period during which the comparative plant may be assumed to develop the business of the going plant is a difficult one, depending largely on the judgment of the appraiser. The writers regret that they have been unable, within the time limit set for

Messrs. Metcalf and Alvord.

this closure, to submit figures relating to the rate of acquirement of business by water-works, although some studies of that sort have been made. The Wisconsin Commission records and a number of the water-works valuation cases, however, furnish data from which curves or diagrams may be constructed, showing graphically the life history of the plant. In this connection the experience, in the development of business, by the new water-works plant of the City of New Orleans will prove of great interest to the student, and up to the present time has seemed to bear out, in a striking manner, the position taken by the writers.

The rapidity with which services can be installed has never seemed to the writers a fair criterion of the rate of development of going value. As bearing on the rate of installation of these services it is interesting to note that Mr. Earle has installed more than 12 000 services per year at New Orleans. It should be remembered, however, that the existing ordinance with regard to the substitution of the public water supply for the cistern has undoubtedly had substantial influence.

Mr. Buerger and Mr. Burns are undoubtedly right in stating that it is possible to construct a gross-income curve, based on new assumptions by the appraiser, as to reasonable income for the works in question, but it may well be asked whether a value so derived represents the going value as of the date of valuation of the existing plant. It may be that the determination of the going value by the writers' method will result in excessive going value, which, added to other elements of value, would give a final value clearly excessive. This simply means that, in the final review of the problem and of the results obtained by the different yardsticks used, the fair-minded appraiser will apply the necessary correction, and it may be very helpful to recompute the going value on the basis of assumption of such a revised set of figures, as an aid in forming judgment, but the writers are of the opinion that such an analysis would be excluded by the Court, even though it admitted an analysis made along the lines suggested by them, unless it were submitted as supplementary to the latter.

In answer to Mr. Buerger's criticism of the writers' assumption of 4% and 6% rates, in their analysis, attention is recalled to their statement that the rate to be used would be modified by many local considerations, and would have to be determined by the appraiser in the light of all existing conditions.

The writers cannot agree with the position of Mr. Burns. His idea of assuming a curve of gross income, representing the appraiser's view as to reasonable income, seems to be quite as speculative as the method suggested by them. As previously stated, neither can they

accept his view that "the rate of installing plumbing fixtures should determine the rate of acquisition of income." Messrs. Metcalf and Alvord.

The fairness of the discussion by Mr. Stearns will be clear to every reader, and seems to call for little comment on the part of the writers. It is perhaps fair to say, however, that their experience in at least one large plant which had gone through more than one reconstruction period, did not indicate that the application of the views herein outlined led to unreasonable results. Had it done so the writers would not have hesitated, in the review of the property as a whole, to have applied the necessary correction to give a final value which, in the light of all existing conditions, seemed reasonable, from the public, as well as the private, point of view. In regard to Mr. Stearns' statement as to the effect of an annual review of the rates and their modification to meet the changing conditions of investment or conditions of the property, it need only be said that such annual review is most unusual, if not without parallel, and, as the writers have frankly admitted, when the time shall come that the financial affairs of public service corporations are open to public service commissions or to the Government, and the rates are modified annually to meet the changing conditions, undoubtedly methods of valuation will approach the standard of actual cost of the service, including therein a reasonable return to the investor. As bearing on the failure of extensions to earn a full and fair return on their cost, the figures presented by Mr. Frank C. Jordan, Secretary of the Indianapolis Water Company, before the New England Water Works Association, in 1910, are significant.

The limits to which this reply has already grown seem to forbid the writers from attempting to take up all the individual points raised by those who have contributed to the discussion, interested as they might be in so doing.

The writers desire to express their appreciation of the interest which the discussions have added to this paper, and in conclusion emphasize the fact that going value is but one step in the process of determining reproduction cost; and the determination of reproduction cost itself is but one step in the process of valuation.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1200

ADDRESS AT THE 43D ANNUAL CONVENTION, CHATTANOOGA, TENNESSEE, JUNE 13TH, 1911.

By Mordecai T. Endicott, President, Am. Soc. C. E.

This day convenes the Forty-third Annual Convention of the American Society of Civil Engineers. We, members, who are familiar with the history of the Association, do not forget that the word, "Forty-third," does not measure its life, for it was founded in 1852, and this is the fifty-ninth year of its existence, and the eve of its three-score.

We assemble again in this delightful city, so beautifully set by Nature, so surrounded by scenes which charm the eye and places of historical interest which people the plain and mountains, after a lapse of twenty years.

The procession of years of the Society is becoming a long one; and when we prefix to it the quarter of a century immediately preceding its organization, which takes us back to a period still within the lives of men now living, we have compassed one of the most marvelous ages in the history of the world. Think for a moment of conditions in 1827, the beginning of this era, and those that obtain at the present time; the development of the steam engine and its application to the many purposes of man, such as the wonderful transportation systems of the world of which such great examples exist in this country; the discovery and uses of the properties of electricity; the telegraph; the telephone; wireless telegraphy; and now the navigation of the air; and so many discoveries of truth and new relations made by science and philosophy. It is only necessary to refer to these great achievements to fill the mind with wonder and awe at the extent to, and manner in which, man has discovered and harnessed and used the

forces of Nature. What a magnificent day of accomplishment this is, and how much the engineer has contributed to the transformations which have taken place and which are still going on! What great things were done in the last century, and with what splendor are we beginning the new! Consider in these things the hand of the civil engineer, the mechanical engineer, the electrical engineer, the hydraulic engineer, the sanitary engineer, and may I not say "the aerial engineer"?

The engineer is in the van in these days of progress, and he lays the whole world tribute to his purposes. Since the air is now being successfully navigated, we may say that he has conquered the heavens, the earth, and "the waters under the earth." The handiwork of the engineer confronts us at every turn. His vocation is one of the greatest in its purposes, one of the greatest in the character and magnitude of its works, and one of the greatest in the qualities of mind, and skill and learning it demands for its successful pursuit.

One of the definitions of civil engineering is "the construction of works of public utility," and this definition applies fairly well to all branches of engineering. I like it better than some of the more labored ones which have been proposed from time to time, and as you know there have been many fruitless efforts to find one that is short, comprehensive, and correct. It expresses the result, for everything the engineer does is directly or indirectly the production of works for the convenience, wealth, comfort, health, safety, or happiness of mankind. If he destroys anything it is only that something better for the public may be constructed or established in its place. What a privilege, what an opportunity is presented in these days for taking part in the accomplishment of these great public utilities! It seems to me that every engineer must be a patriot, a humanitarian, an altruist, if you please, for all his works have for their aim and end the security, well-being, prosperity, and happiness of man; and this cannot be said of many callings in life.

The discovery of these new forces, the study of their possible physical applications, the extensive and valuable inventions which have followed, and their innumerable applications to the industries of the world, especially in new countries like our own, where there exist such treasures in the forest, the field, and the mine, have brought about a wonderful degree of progress and development in all departments of industry. The transformation is almost startling. We are in a great industrial and commercial age, and the prime agencies in its vast development and progress are steam and electricity; these in the hands of man are making advancement phenomenal, and have brought the people of the world face to face with new problems in commercial affairs. They have been largely the instruments which have brought about the great combinations of capital and brains in

the great corporations of this day. Their uses have been so perfected and enlarged by the engineer that the tendency has been to bring closer relations in the business world. This has operated under natural laws, and the results may be regulated, but never destroyed.

So absorbed has the world become in industrial and commercial enterprises, so great is the mad rush for control and wealth, and so marked has this become in our own country where the resources to be developed are so vast, that many of the ablest minds of the day do not view them with unalloyed pleasure. We are sometimes reminded that the age has become one of materialism largely to the exclusion of the pursuit of culture and art, that we are so busy with the scientific side of life and the pursuit of wealth that art languishes, and that accordingly we lose much of the richest and best of life. Others tell us that in this industrial age, with its accumulations of wealth, culture is advancing more rapidly than wages, not merely the wages of the mechanic and laborer, but the compensation of all employees.

An educator has recently proposed as a remedy for such a social crisis, an education that increases efficiency and augments the income, philosophy and poetry to give way to economic facts. I admit the force of this argument as applied to certain employees, and I believe there is a place for the vocational school, and that in many situations there should be more economic and industrial training and less of philosophy, etc. Another educator, a French Academician of distinction, takes a position quite at variance with the one just referred to, and deplores the fact that secondary education is threatened on one hand by primary education and on the other by higher or professional education, youths in too many cases are going directly from elementary classes to the professional schools, and becoming what he denominates "skilled artisans" in the different professions.

Certainly there are limitations in either direction, but I have often felt that this busy, industrial age has had an influence to restrict the proper education of the engineer, and confine it too closely to the immediate demands of his practice; and it is because of this tendency that I have made mention of the great material interests that center about the engineer, and of the magnitude and high character of the tasks that are committed to his hands, as bearing upon the question of the proper preliminary preparation therefor; for certainly these relations point with emphasis to the responsibility of the engineer in qualifying himself for his task, and of the technical schools of the world to give to it the highest type of engineer. This responsibility is clear whatever may be our individual ideas of what is necessary to produce that type.

But I think all will agree with me that wherever practicable the education of the engineer should be broad, both with respect to its general as well as its professional features. Our schools of engineering

should aim to send out men of the most extensive and thorough education. It is not always that the technical school richest in equipment does the best work, nor is it always the school that includes a large amount of practical instruction and work in its curriculum. I think it is an easy matter to overestimate the value of practical instruction, and put too much time and money into it. The first and paramount duty is to give the young engineer a thorough, broad, and exhaustive theoretical education. If he has this basis to build upon, the practice will soon take care of itself. The practice comes, that is what he goes out into the world for, but you cannot then count upon him giving much time to the acquirement or completion of the broad theoretical education he ought to have. Many are not so inclined, and many never find the necessary time when once launched into the busy whirlpools of these generations. I would say, give the most thorough and the broadest theoretical education possible, and if any paring must be done, let it be on the practical side; principally for the reason that it is the bulk or the mass of the men sent out that must be considered, for, taking out a few exceptional cases, they will never afterwards attain to the right standard, and in the exceptional cases it will be reached at the expense of much time and with much disadvantage.

When practicable, the engineers should have a liberal culture in the natural sciences and in general literature, to fit them well for coping with the great practical and economic questions which require specially trained minds, minds which should be enriched in cognate subjects to give them a closer and more intimate touch with the secrets of the world, to quicken their powers of observation, to enlarge their minds, to amplify and strengthen their powers of mental assimilation, and heighten the quality that enables men to discern accurately the relations between cause and effect, and the inter-relations of all the forces in Nature, all of which is so important to the engineer. Engineers should be accomplished scientists. They should be something more than the average acceptation of the meaning of the title "engineer," and as far as possible their minds should be instructed and disciplined in all that goes to make up the cultivated engineer. An education of this scope rounds out the man in all directions, and in enlarging his perceptions and vision, cultivates ideals and imagination. We do not consider imagination, ordinarily, as an important quality in the engineer, and yet I think no engineer ever rises to eminence in his profession without it. Ex-President Eliot, of Harvard, one of the most eminent educators in the world, says it is just as important to the engineer as to the artist and literary man, and that the engineer should stand with his feet on the ground and his head in the skies.

I am not unmindful of the fact that the engineering school is not absolutely necessary to an engineering education. The history of the

profession refutes a contrary theory. Its pages are replete with the splendid accomplishment of engineers who were not trained in the schools; and the annals of this Society contain the records of many eminent members who were wholly without the aid of technical institutions, but who were profoundly versed in the theory as well as the practice of their profession; and some of these men were highly educated in subjects outside the limits of their vocation. There were no engineering schools in the earlier days, and the best they could obtain was a good grounding in mathematics and the natural sciences in the academic schools; but in many instances the difficulties encountered served to increase the zeal and avidity with which they pursued their object, and magnified the value of their accomplishments, so true is it that what we work hardest for we appreciate and prize most.

However the engineer fits himself for his work, whether by study in the schools or by private study, his individual responsibility remains the same, to accomplish it in the broadest and best manner possible, not only for his own sake, but for the profession he represents, and for the public or other interests he serves. He must do his part in building up and maintaining a standard and *esprit* worthy the duty to which his profession is called in this Twentieth Century. In the evolutions of the past 75 years his profession has arisen to be one of the most important in the world; indeed, the commercial and industrial interests are dominant in the world to-day, and the engineer stands at its pulse. The world, and I may say this country especially, has a magnificent future, and it is going to be in a very large measure what our profession makes it, and it is the high privilege and duty of the engineer to be, in so far as in him lies, worthy of that high calling, and to make his profession fully equal to it. The President of one of the great engineering societies of Great Britain, not long since, said, "The engineer is the salt of the earth." Now, if he be that salt, he must have the savor.

Responsibility rests not only in the school and the individual. Engineering societies have a responsibility in raising the standard of engineering education and of the profession. What public sentiment demands in the schools and in the profession will be forthcoming, and this Society with its membership of more than 6 000, scattered over this country and in other parts of the world, can wield a healthful influence in these directions.

This Society should be open to all young men of good character who sincerely and earnestly enter upon the profession, but the entrance to the grades above the Juniors, whether by original admission or by advancement, should be guarded with great discrimination, and the high honor of full membership in the American Society of Civil Engineers should be accorded only to those who attain a high standard

of personal as well as professional character, and who have reached an advanced degree of excellence in the theory and practice of the profession, as exemplified in a length of important service and high character of work accomplished, and all standards set should be enforced with the utmost strictness; the doubts should always be solved in favor of the Society.

In this way is it practicable for this great Society to do much to create and maintain a high plane for the profession, and fulfill the object laid down in its Constitution, "the maintenance of a high professional standard among its members," and in so doing it will render a high service to the public.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1201

SINKING A WET SHAFT.*

By John P. Hogan, Jun. Am. Soc. C. E.

With Discussion by Messrs. Mason D. Pratt, Charles B. Buerger, L. White, H. M. Hale, H. F. Dunham, T. Kennard Thomson, Bertrand H. Wait, T. H. Wiggin, V. H. Hewes, and John P. Hogan.

On the new Catskill Aqueduct for New York City some of the most difficult problems, both in design and construction, have been encountered in the deep pressure tunnels or siphons. The design and location of these tunnels have been described in the technical press.† The first siphon to be placed under contract was that crossing the Rondout Valley, near High Falls, N. Y., a work second in importance only to the Hudson River crossing, and now more than half completed.‡

There are eight shafts on the Rondout Siphon, varying in depth from 374 to 710 ft. (three permanent shafts and five sunk to expedite the construction of the tunnel), and all, excepting Shaft No. 4, were completed to tunnel grade without any great difficulty. It will be noted on the profile, Fig. 1 and Plate XXIX, that the tunnel between Shafts Nos. 3 and 5 passes under Rondout Creek and its buried preglacial gorge, and, due to a sharp steepening of the dip, also passes through a number of different rock strata. The examination of exposures of these rocks at other points (and preliminary surveys and borings) indicated that some of them were very hard while others

* Presented at the meeting of May 3d, 1911.

† *Engineering Record*, January 29th and February 5th, 1910; and *Proceedings*, Municipal Engineers of New York City, 1909.

‡ A description of the power-plant for this work was published in *Engineering Record*, April 10th, 1909; and a description of the methods of tunnel excavation on January 1st, 1910.

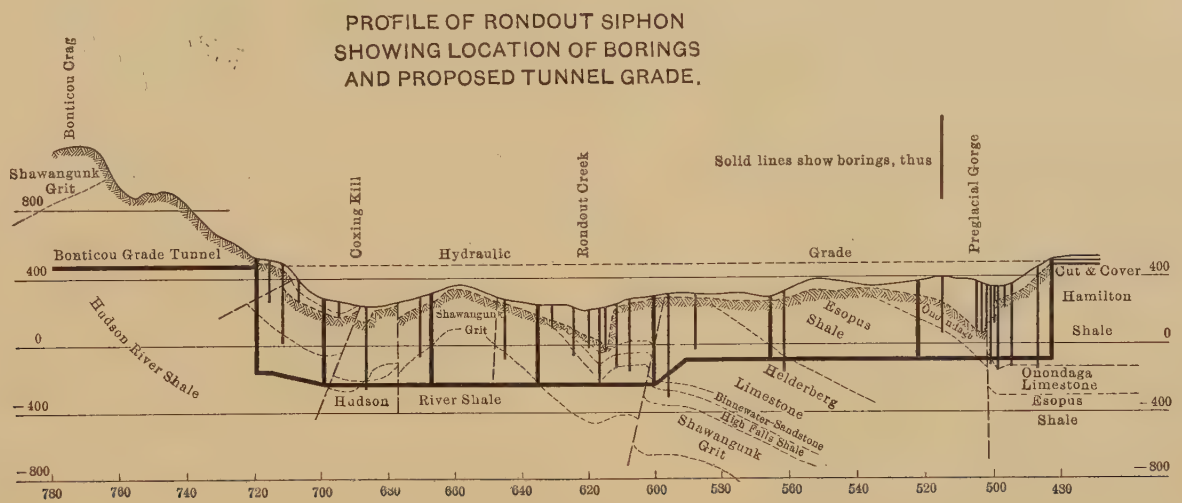

PROFILE OF RONDOUT SIPHON
SHOWING LOCATION OF BORINGS
AND PROPOSED TUNNEL GRADE.
Bonticou Crag
Shawangunk Grit
800
Bonticou Grade Tunnel
400
0
−400
−800
Hudson River Shale
Coxing Kill
Hydraulic
Rondout Creek
Grade
Solid lines show borings, thus
Preglacial Gorge
Shawangunk Grit
Hudson River Shale
Esopus Shale
Onondaga Limestone
Helderberg Limestone
Binnewater Sandstone
High Falls Shale
Shawangunk Grit
Cut & Cover
Hamilton Shale
Onondaga Limestone
Esopus Shale
780
760
740
720
700
680
660
640
620
600
580
560
540
520
500
480

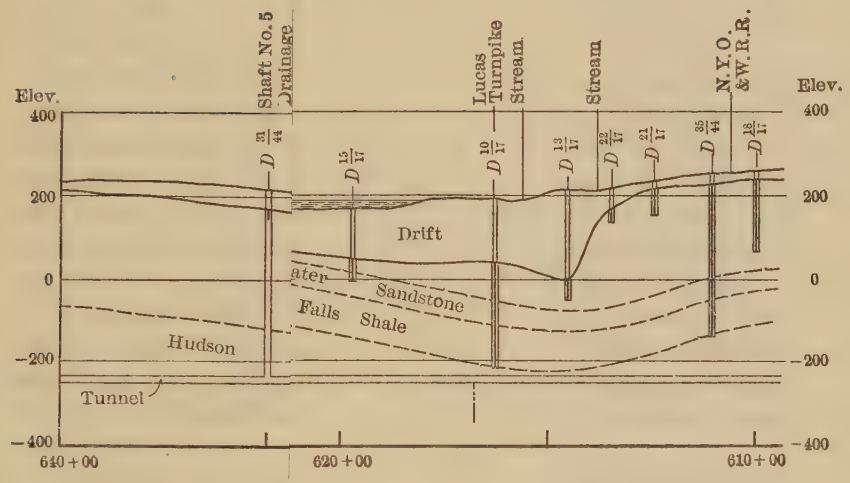
FIG. 1.

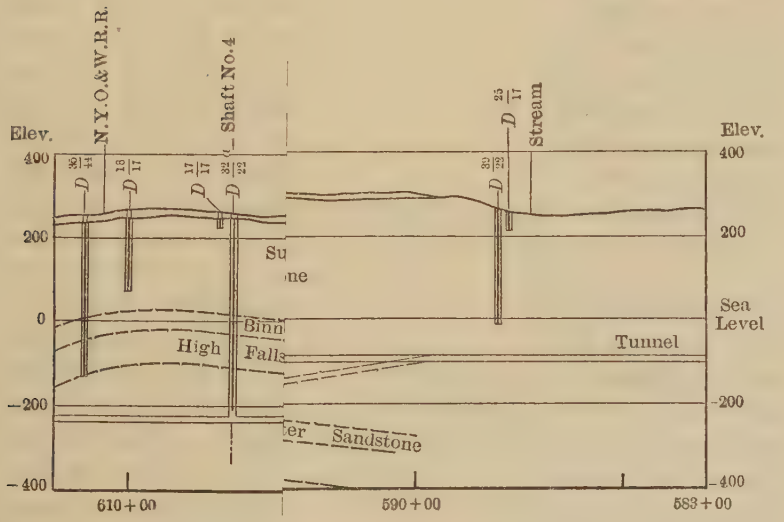
were of a porous, water-bearing nature. It was evident from the start, therefore, that this would be the most difficult part of the work, and that an intermediate shaft would be required in order that the progress might be kept equal to that in other parts of the tunnel. Any location of the shaft would require it to penetrate the strata in which the trouble was expected, and, for various reasons, the choice was limited to the stretch between Stations 600 + 00 and 607 + 50, as shown on Plate XXIX.

Diamond-drill borings were put down in the vicinity to define the strata, and at Stations 600 + 00 and 607 + 50, 4-in. shot-drill borings were made to permit of pumping experiments by which it was hoped to determine the relative porosity of the rock. An oil-well rig, having a capacity of about 90 gal. per min., was used in these experiments. The ground-water elevation was about 30 ft. below the surface at both holes. At Station 607 + 50 the discharge was about 50% in excess of the theoretical capacity of the pump, reaching a maximum of 130 gal. per min., and, although 1 071 000 gal. were pumped, it was impossible to lower the elevation of the ground-water. At Station 600 + 00 there was no excess in discharge over the capacity of the pump, and the ground-water level was lowered 53 ft. by pumping 2 020 000 gal. The surprising part of this experiment was that the ground-water level at Station 607 + 50 was also lowered 15 ft., although previous pumping at that point had had no effect.

The water at both points was strongly impregnated with sulphur, and, while there was no thoroughly satisfactory explanation of these phenomena, the theory was advanced that at Station 607 + 50 gas was liberated by the churning action of the pump and thus increased the pressure, forcing the water through the working barrel of the pump. The records of the borings showed that the water was first encountered in the Binnewater sandstone. This entire bed is exposed in a vertical cliff below the High Falls, on Rondout Creek, less than a mile from the tunnel line, and examination showed it to be a soft, coarse-grained sandstone with numerous porous layers, probably caused by the solution of calcareous material. The High Falls shale immediately below it also showed considerable porosity, and the indications were that the shaft would be a wet one. The location at Station 600 + 00 seemed to promise a better shaft, but that at Station 607 + 50 gave a reduced length of tunnel in difficult rock. Both locations were shown in the

PROFICINITY OF SHAFT No. 4.





contract drawings, and the contractor was given the option of choosing either one. The contractor elected to sink at Station 607 + 50. The result of the pumping experiments was to include in the contract provisions for an emergency pumping plant having a capacity of 1 800 gal. per min.

The total depth of the shaft is 498 ft. and the strata penetrated were: 6 ft. of glacial drift, 226 ft. of Helderberg limestone, 39 ft. of Binnewater sandstone, 92 ft. of High Falls shale, and 134 ft. of Shawangunk grit. The Helderberg limestone is a hard, dark gray rock, with layers of flinty material and few seams; the Binnewater sandstone is a soft, greenish, fine-grained rock of a high general porosity, and with numerous open water-bearing seams; the High Falls shale is a soft rock, thin-bedded in horizontal layers, with numerous seams, and varying in color from light green to red; the Shawangunk grit is an exceedingly hard, white, quartz conglomerate. The shale and sandstone grade into each other very gradually, the latter containing numerous shaly layers and *vice versa.*

The contract was let on June 12th, 1908, and excavation at Shaft No. 4 was started on July 20th, 1908, air being supplied from a temporary plant. Less than four months later, air was turned on from the large central plant.* The shaft, being for construction purposes only, is rectangular, 10 by 22 ft. over all. It is timbered throughout with 10 by 10-in. wall plates, 6 by 8-in. buntons and 2-in. lagging, making its size 8 by 20 ft. inside the timbers, and is divided into three compartments. Four No. 12 Tandem Cameron sinking pumps, each having a capacity of 450 gal. per min., and one No. 10 having a capacity of 300 gal. per min., were ordered for the emergency pumping plant, and, pending delivery, several No. 7 and No. 9 Cameron sinkers were kept in reserve.

The shaft was dry to a depth of 80 ft., but, on September 2d, the water rushed in from the bottom through the 4-in. bore-hole at an estimated rate of from 600 to 800 gal. per min., and the shaft was filled to within 40 ft. of the top. As the emergency plant had not then been delivered, a simple air-lift was installed which at first threw 1 600 gal. per min. As the efficiency decreased, due to lack of back pressure, two more stages were added, and, with the aid of two No. 9 sinking pumps, the water was lowered to within a few feet of the

* Described in *Engineering Record*, April 10th, 1909.

bottom. A 4-in. swedged nipple was then driven into the hole, and casing was attached. It was decided to grout the bore-hole, and a 1-in. pipe was let down inside the hole to the grit and extended to the surface. The shaft was then allowed to fill up, to equalize the pressure and prevent flow, and 1:1 grout was poured down the 1-in. pipe, while the latter was raised gradually. When the hole was filled to the top with grout, the 1-in. pipe was withdrawn, the shaft was unwatered, and sinking was resumed.

From this point to the bottom of the limestone the shaft was so dry that occasionally water had to be carried down to mud the drills. It was realized, however, that a large quantity of water under heavy pressure was present in the lower strata, and serious doubt began to be felt as to the possibility of sinking this shaft by ordinary methods. J. F. Sanborn, Assoc. M. Am. Soc. C. E., Division Engineer of the Peekskill Division, Northern Aqueduct Department, then directed attention to the recent successful use of the cementation or grouting process of sinking.* The contract called for grouting behind the lining of the completed tunnel, with a pressure of 300 lb. per sq. in., and the delivery of this apparatus was hastened.

At a depth of 215 ft. the shaft entered the Binnewater sandstone, and in this formation the water gradually increased until, at a depth of 260 ft., the shaft was making 225 gal. per min. This was handled by two No. 9 Cameron sinkers. While the sump was being drilled, on December 20th, 1908, an additional flow of about 600 gal. came in through one of the drill holes under considerable pressure, quickly drowned out the sinking pumps, and flooded the shaft to within 70 ft. of the top. Preparations had been made to plug any water-bearing hole, but the flow was of unexpected force and volume. It is interesting to note that the water, at this time and later when the shaft was flooded, rose to a lower elevation each time, indicating that the pumping had considerable effect on the general water-table.

A long delay now ensued before sinking was resumed. A single-stage lift, having an 8-in. discharge and 2-in. feed, lowered the water to within 60 ft. of the bottom, throwing from 600 to 350 gal. per min. A second stage of the same size was then installed, discharging into the first 100 ft. from the bottom. This had an initial capacity of 500 gal. per min., and lowered the water to within 30 ft. of the bot-

* Described in the *American Engineering and Mining Journal*, August 1st, 1908.

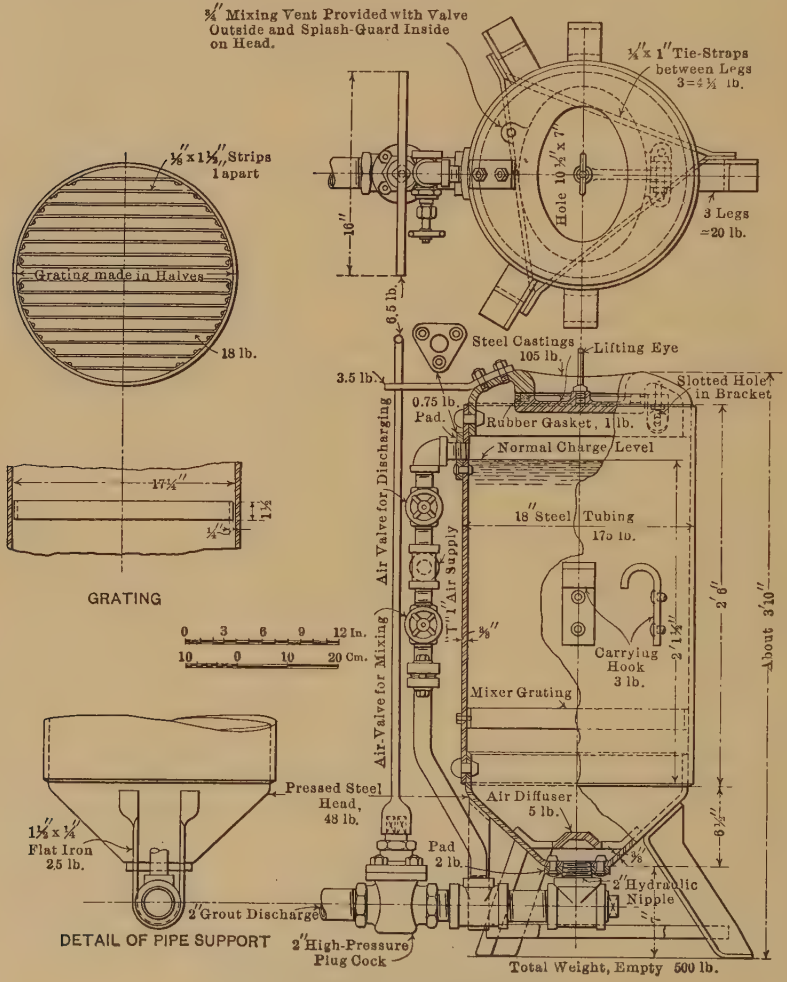
tom. A third stage was added, and, with the aid of two No. 12 Cameron sinkers, each having a capacity of 450 gal. per min., the bottom was unwatered on January 24th, 1909, the No. 9 Cameron pumps were recovered, and the hole was plugged with a 2-in. swedged nipple and gate-valve. A gauge on the pipe indicated a pressure of 75 lb. Drilling was resumed for the purpose of finding the location and extent of the water-bearing seams, the holes being plugged with pipes and valves as water was encountered. On February 3d, 1909, while one of the pumps was temporarily out of commission, two of the holes struck water. The discharge hose broke on the second pump, when speeded up, and the shaft was again flooded and the pumps drowned. This was the beginning of a series of troubles with the pumps which will be described in detail later. Due to their great weight, the high head, and the consequent force of their pulsation, these pumps gave continual trouble with their discharge lines, and discharge and suction hose pipes. These troubles were minimized later by fastening them to the timbers and by improvements in the quality of the equipment, but the sinking pumps could never be relied on for satisfactory service on heads greater than 200 ft.

The pumps were recovered by using the air-lift, and an additional No. 12 pump was lowered into the shaft. On February 10th, before the holes in the bottom could be plugged, the pumps again went out of commission in rapid succession, the shaft was again flooded, and the pumps were submerged. Again the pumps were recovered by the air-lift and the two No. 9 Camerons; the bottom was uncovered and the holes were plugged on February 16th.

Drilling in the bottom was resumed through pipes and gate-valves which could be closed quickly. The leakage from the sides and around the pipes in the bottom had now risen to 450 gal. per min. With the discharge hose and gaskets on the discharge line blowing out above, and water spurting in from the bottom, difficulty began to be experienced in getting labor. On February 21st the shaft was again flooded, due to pump troubles, and was not recovered by the air-lift until March 12th.

A sufficient number of holes had now been drilled to indicate that the rock for at least the next 8 ft. was soft and porous with several open seams of a maximum width of 8 in. The largest seam was at one point within 18 in. of the bottom of the shaft. The behavior of single 2-in. drill-holes made it unpleasant to contemplate the quantity

of water which might be encountered if sinking were resumed. Accordingly, it was decided to attempt first to cut off the water by grouting, a matter which had been under advisement for some time. As the rock lay in horizontal layers, it was hoped that the water would not



TANK GROUTING MACHINE.

FIG. 2.

follow the shaft down, but could be completely cut off by grouting the porous layers.

A battery of four Canniff tank grouting machines, Fig. 2, was set up at the top, with a 2½-in. pipe down the shaft and a 2-in. hose con-

nection at the bottom. At first the grout leaked back into the shaft around the pipes and through seams in the bottom. It was thought that a concrete blanket might be necessary, but the difficulty of placing it would have been great. After several experiments, finely-ground horse manure was injected with the grout. Although there was considerable waste of grout, the manure finally clogged all openings, and the bottom was made tight. In three days 2 900 bags of cement were forced in, and the holes were then blocked. The large quantity of cement indicated the size and extent of the seams. After the grout had set, additional holes were drilled, and struck water under a pressure of 65 lb. at a depth of 14 ft. These holes were then grouted, 60 bags of cement being used.

Experience, up to that time, indicated that the sandstone was very porous and water-bearing, and the High Falls shale lying underneath was believed to be little better. In view of the success of the first grouting, it was decided to put diamond-drill holes down to the top of the Shawangunk grit, and to attempt to grout the intervening strata. The distance from the bottom of the shaft to the grit was 100 ft., and it was decided to bore six holes. At first, 3-in. holes were drilled to a depth of about 5 ft., swedged nipples with gate-valves were driven in tightly, and 2½-in. casing was extended to the top of the shaft where the diamond drills were set up. Three holes were drilled by a Sullivan Badger machine taking a 1-in. core, and three with a Sullivan "C" machine taking a 2-in. core. Water was encountered at a depth of 50 ft. The holes were continued to the grit and 175 bags of cement were forced in under a pressure of 275 lb. The small quantity of cement required was looked on by some as an encouraging sign of better conditions; to others it seemed to cast doubt on the efficiency of the grouting, when considerd in connection with the depth and small diameter of the holes. Both beliefs were shown to have a measure of truth by later developments. The shaft was now making 225 gal. per min., all coming in from the sides.

Sinking was resumed on April 25th, 1909, after a delay of four months, and, in the first 15 ft., numerous seams were passed with a maximum width of 8 in. All were filled with grout, and were tight. Below this point no large seams were encountered, but the seepage from the sides increased until the inflow was 350 gal. per min. at a depth of 280 ft. Trouble with the sinking pumps had become so frequent,

due to the high lift, that it was decided to put in a collecting ring and establish a station at a depth of 265 ft. A sketch of a typical ring is shown on Fig. 3. The pumps on the bottom discharged into the ring, and the water was relayed to the top by two No. 12 Cameron sinkers, which were protected from blasts by a bulkhead of timbers. Sinking was resumed, and when the shaft had reached a depth of 300 ft. and was making 450 gal. per min., a second ring was installed with one No. 12 Cameron sinker pumping to the surface. The second ring intercepted about 100 gal. per min., the upper ring still being used as a station for the two No. 9 Cameron sinkers on the bottom. As the excavation proceeded, a 2-in. seam was encountered, increasing the inflow to 525 gal. per min., and a third ring was added. The water was strongly charged with sulphur; and the H_2S gas, which had been giving more or less trouble, now increased to 9 parts in 10 000. The water caused cuts and abrasions in the flesh to develop into running sores, and the gas, in addition to a general debilitating effect, irritated the eyes greatly, causing temporary blindness. Great difficulty was experienced in getting men, and it was necessary to raise the pay and shorten the hours of work. The bottom was flooded several times, but the water never rose higher than the first ring.

The shaft was now (July, 1910) about 320 ft. deep, and was so full of pumps and their air and discharge lines that it was almost impossible to do any work. A step which should have been taken earlier, but had been postponed by a hope of improvement in the rock, was now decided on, namely, a station chamber in the side of the shaft.

On July 15th the pumps at the upper ring broke down and the shaft was flooded for the sixth and last time. The shaft was unwatered, the six pumps at the bottom were recovered by the invaluable air-lift, and the excavation of the chamber was started on August 1st at a depth of 309 ft. The chamber, as shown on Fig. 2, was 17 by 24 ft. and 10 ft. high, and had a sump 16 by 22 ft. and 5½ ft. high, having a capacity of 14 500 gal. Three 24 by 10 by 20-in. Cameron horizontal condensing pumps, with a combined capacity of 1 050 gal. per min., were installed in the chamber. To relieve the heavy load on the power-house, it had been decided to run these pumps by steam, and three 100-h.p. boilers were set up at the top of the shaft. A 4-in. steam line from the boilers to the pumps was wrapped successively with asbestos, felt, and tin, to prevent the condensation of the steam and

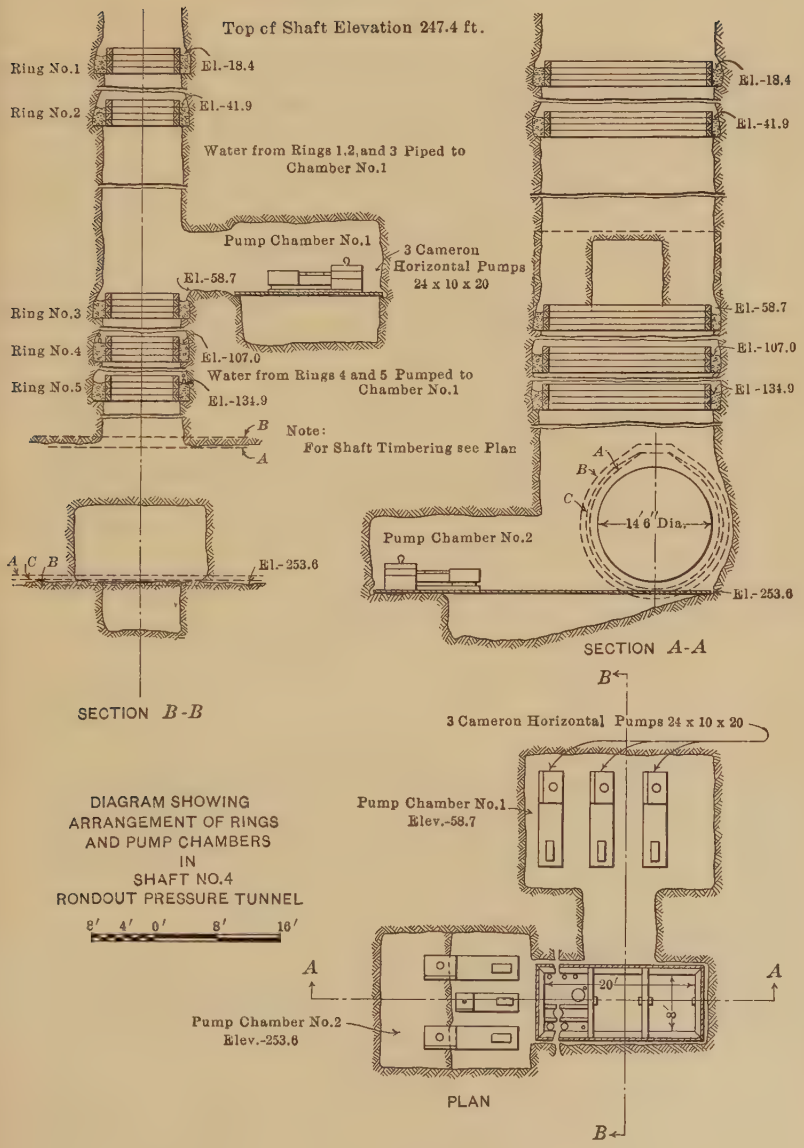
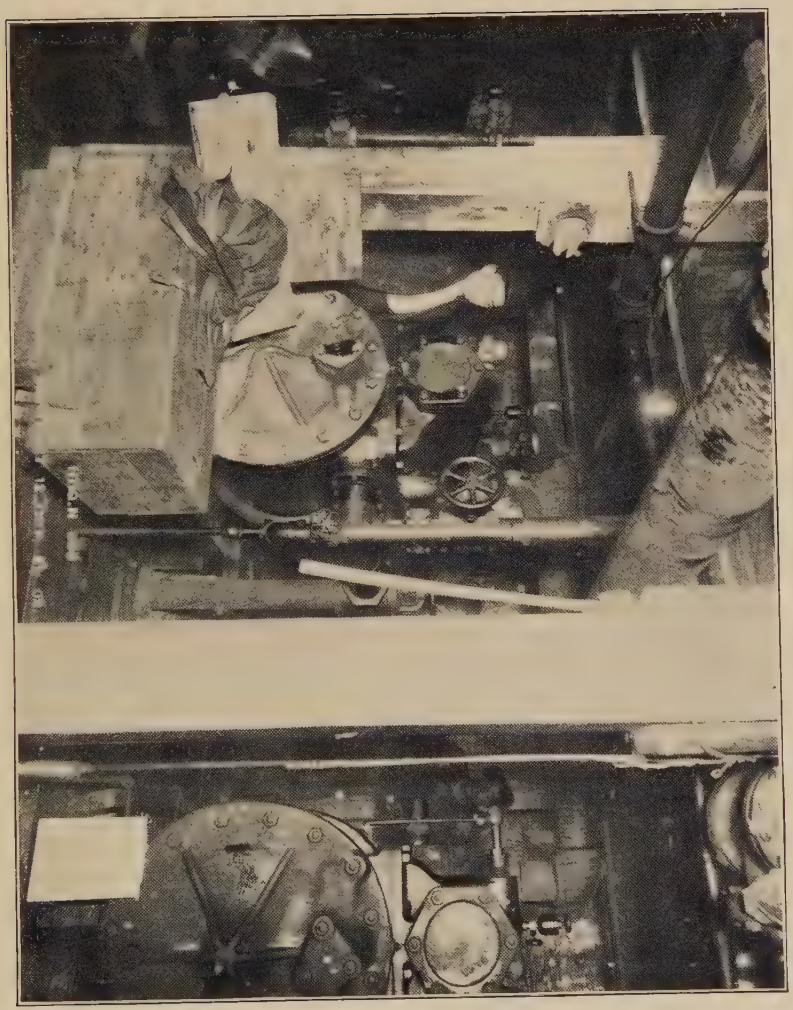

Top of Shaft Elevation 247.4 ft.
Ring No.1
El.-18.4
Ring No.2
El.-41.9
Water from Rings 1,2, and 3 Piped to Chamber No.1
Pump Chamber No.1
3 Cameron Horizontal Pumps 24 x 10 x 20
El.-58.7
Ring No.3
Ring No.4
El.-107.0
Ring No.5
Water from Rings 4 and 5 Pumped to Chamber No.1
El.-134.9
Note: For Shaft Timbering see Plan
Pump Chamber No.2
14'6" Dia.
El.-253.6
SECTION A-A
SECTION B-B
3 Cameron Horizontal Pumps 24 x 10 x 20
DIAGRAM SHOWING ARRANGEMENT OF RINGS AND PUMP CHAMBERS IN SHAFT NO.4 RONDOUT PRESSURE TUNNEL
8' 4' 0' 8' 16'
Pump Chamber No.1 Elev.-58.7
Pump Chamber No.2 Elev.-253.6
20'
PLAN

FIG. 3.

fog in the shaft. A water-bearing seam was encountered in the chamber, increasing the inflow to 725 gal. per min., and during the excavation the number of pumps in the shaft increased to ten, of all sizes and styles. By this time the gas had become so bad that it was necessary to ventilate the shaft. Three Sturtevant blowers (two No. 35 and one No. 45) were installed at the top, with 10-in. and 14-in. pipe lines. A mixture of 1 part of chloride of lime and 20 parts of ordinary lime was also sprinkled continuously down the sides of the shaft. After steam was turned on, nearly two weeks were consumed in removing superflous pumps, air-lines, and discharge pipes from the shaft. All water from the rings was either pumped or led by gravity into the sump at the chamber.

Sinking was resumed on September 9th, 1909 (a pilot drill-hole being kept well below the bottom) and all seams were grouted in advance, none of them taking more than 50 bags of cement, until the grit was reached. There was still some seepage from the sides, which sprayed in the shaft and increased the bad effects of the sulphur and gas. Several rings with small horizontal pumps were installed to prevent this. At the contact with the grit an additional flow of 125 gal. per min. came in through the old preliminary drill-hole which had been previously grouted, increasing the total flow to 850 gal. per min. This hole was plugged with a nipple and gate-valve, and registered a pressure of 95 lb. The flow was cut off by forcing in 348 bags of cement under a pressure of 275 lb. All water-bearing seams in the grit were grouted, the maximum quantity of cement used at any point being 100 bags. Sinking was continued until December 31st, 1909, when the shaft reached tunnel grade at a depth of 497 ft., and the headings were turned. The elapsed time from the start of excavation was 18 months, an average monthly progress of 28 ft. Exclusive of delays due to trouble with pumps, the average monthly progress was 45 ft. The highest monthly progress was 80 ft., which was made in the limestone. The total quantity of water pumped during the sinking was 86 181 000 000 ft-gal., and the total quantity of cement used for grouting was 971 bbl.

The general method of excavation was to sink about 65 ft. below the timbers and then place about 50 ft. of timber, the lower dead-logs being about 15 ft. above the bottom of the sump. After large flows of water were encountered, it was found necessary to keep timbers close



PUMP CHAMBER IN SIDE OF SHAFT No. 4, 309 FT. BELOW THE SURFACE.

to the bottom, for convenience in handling pumps and hanging them and their air and discharge lines. Wire guides were carried down to the bottom of the timbers, and a cross-head or "billy" was used with the bucket. Excavation was made with a center cut or sump and two side or bench cuts, by the continuous method, drilling and mucking proceeding simultaneously, as indicated in Fig. 4. Drilling was started on the cut with four 3½-in. Ingersoll drills mounted on two quarry bars, as soon as the bench muck had been cleared away. The sump was then shot, and drilling was started on one bench while the sump was being mucked. This bench was shot as soon as the sump had been cleaned out, and drilling was then started on the other bench. The average number of holes for a complete advance was 32, and the average advance was about 7 ft. The pumps were hoisted into the timbers while blasting.

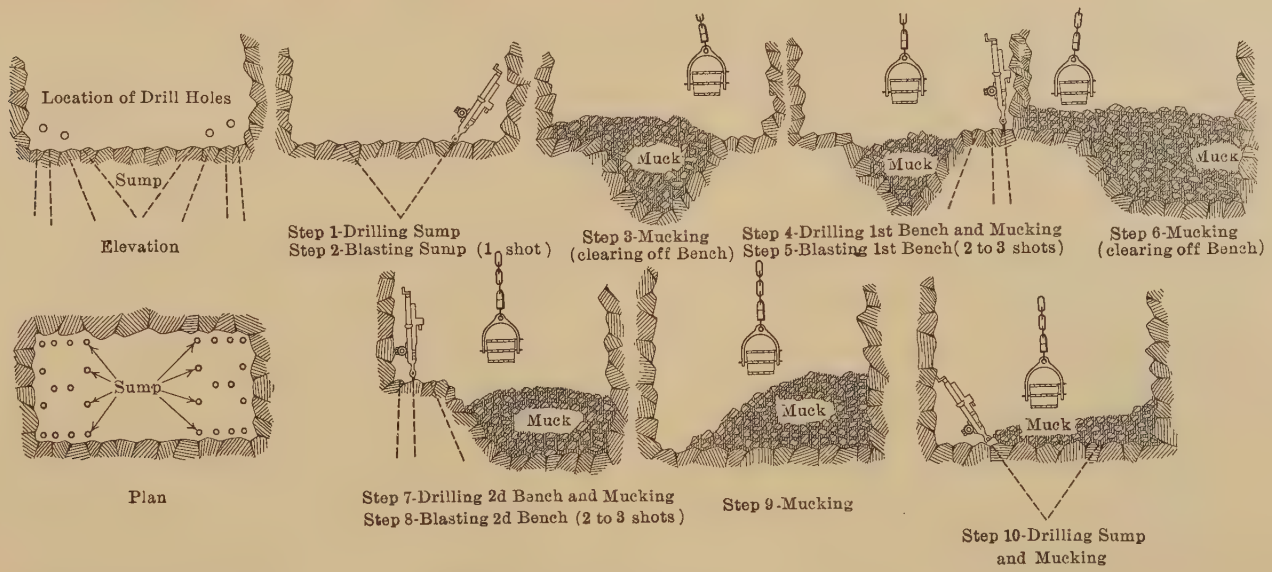
When the inflow became great, the pumps were kept in operation during blasting. The shaft was excavated in separate halves, the pumps being moved to the opposite end of the shaft and protected by a bulkhead. After the station chamber was in operation, the water was kept off the bottom as much as possible by rings, and the former method was resumed. The superintendents and laborers employed at this shaft were mostly trained shaft-sinkers, drawn from the Pennsylvania coal fields.

The methods used in grouting differed somewhat in detail, but in general were as follows: A 2-in. pipe (with gate-valves) was driven into the hole as far as possible. There was great difficulty in getting tight packing around these pipes, and preventing leakage of grout back into the shaft. Wooden wedges, flannel, oakum, oil-well packers, and cement were used freely, but at times it was necessary to resort to the horse manure. A pressure of from 100 to 275 lb. was used, the former being obtained from the main air-line and the latter from a small Westinghouse high-pressure compressor. The grout machines were placed at the top of the shaft and connected with the hole by a 2½-in. pipe. The long pipes clogged somewhat, and cut down the quantity of sand which could be used, but the difficulty of operating the machines at the bottom would have been too great. A battery of from four to six Canniff air mixing tank machines was used, and connected to one pipe. Owing to the simplicity and rapidity of their operation, these machines are admirable for forcing in large quantities of grout,

and kept an almost continuous stream of it passing down the pipe. Between batches, a minimum pressure of 100 lb. was kept on, to hold the grout in place, as the back pressure from the water was at times as high as 95 ft. A very weak mixture was used at first, and the quantity of cement was increased gradually; if this continued to flow freely, sand was added until the pipe began to clog. All grouting was as nearly continuous as possible, and was uniformly successful (except with the deep diamond-drill holes), at one time cutting off a leakage of more than 400 gal. per min. With the diamond-drill holes, it is believed that, in addition to the length and small diameters of the holes, the partial failure was due to the fact that the chips and dust caused by the drilling clogged the seams. Experience seemed to show that, with the stratified rock (as had been anticipated), the water cut off was not forced into the shaft at a lower level. The grout set very slowly and developed little strength, although the waste grout carried out through the pumps finally set outside.

For any head of more than 200 ft. the sinking pumps gave very poor results, for a number of reasons. The main reliance was placed on four No. 12, 19 by 19 by 12 by 16-in. Camerons, with a 10-in. suction and 8-in. discharge. Each pump weighed about 5 tons, and they were so large that they could not be passed by each other in the same compartment. Against high heads they never pumped much more than half their rated capacity, because, when speeded up against the heavy back pressure, the pulsation was so great that it soon tore them loose from their connections. The vibration was so great on a flange-joint discharge line that bolts and flanges broke continually, and it was necessary to replace it by a screw-joint line. One great trouble was the impossibility of making rigid connections with the discharge line. Wire-wound discharge hose, rewound with rope, frequently gave way under the pressure, and the connections between the hose and the pumps were continually failing.

Prior to the installation of the horizontal pumps in the chamber, the air-lift was the one and only reliable part of the unwatering equipment. It was very wasteful of power, but made unnecessary both a large reserve of pumps and the slow and tedious process of staging them down the shaft when it was flooded. It was efficient until the depth of water in the shaft was less than 60% of the total depth of the shaft. When the water was lowered to 40% of the total depth,



Material, Shales and Limestones.
No. Holes Cut 8 x 10 = 80 feet.
Bench, 12 x 8 = 96 feet.
Trimming 8 x 8 = 64 feet.
Total 28 Holes = 240 feet.
Average No. of Holes = 23 for 6⅔ ft. advance.
Average Progress – One advance in 1½ days.

Yardage per foot = 8.15 cu. yd.
Force: 3 shifts of 2 drillers 2 driller's helpers and 10 muckers.
Dynamite per advance = 120 lb.
Average rate per drill per hour.
Quarry or Shaft bar used.
Cut drilled, shot and mucked North Bench then drilled, shot and mucked.
South Bench then drilled, shot and mucked. Started drilling bench before mucking of cut was finished.

SKETCH SHOWING VARIOUS STEPS IN CONTINUOUS METHOD OF SHAFT SINKING AS USED AT SHAFTS, RONDOUT SIPHON.

FIG. 4.

the air-lift was usually staged. Sinking pumps were always required to take out the last 10 or 15 ft. of water.

Pumps operated by steam required about half as much boiler-horsepower and half as much coal as for operation by air. When it is considered that there was a central power-plant for delivering air, and that the cost of installation, maintenance, and hauling coal must be considered as an extra, it is probable that the steam plant effected no economy. The principal cause for its installation was an insurance against accident or delay due to any break-down or overloading of the central power-plant. Steam was never used with the sinking pumps, on account of the excessive heat and the fogging of the shaft which would result. The steam horizontal pumps used in the chamber were condensing, thus preventing this difficulty. The air gave considerable trouble by freezing up the pumps, although re-heaters and drips were used.

As a source of difficulty and delay in sinking this shaft, the H_2S gas was second only to the water. It is believed to be due to the decomposition and dissolution of pyrites in the rock, under the heavy pressure. The gas arising from the shaft soon oxidized the paint on a building 100 ft. away, and the discharge water would tarnish silver coins in less than a minute. In addition to their debilitating effect on the men, the gas and water attacked pipes, cables, rubber gaskets, and hose with serious effect.

It is believed that an ultimate economy might have been effected by constructing the chamber earlier, and at a higher level, with perhaps a second chamber in a lower position. The completion of the shaft is due to the successful use of the process of cementation, or grouting. The quantity of water which might have been encountered without grouting is largely a matter of speculation, but it is thought that it would have approached the sum of the amounts encountered at different times. One of the things which lead to this belief is the fact that, while ungrouted inflows near the top of the shaft showed a very slow and steady decrease due to the general lowering of the water-table, the rate was apparently unaffected when new flows of from 300 to 400 gal. per min. were encountered at the bottom. Another is the fact that an inflow of about 2 000 gal. per min. was encountered in the same strata in the tunnel, between Shafts Nos. 3 and 4.

All water pumped from this shaft was paid for under the contract

FIG. 1.—HEAD-FRAME AND WATER-MEASURING DEVICE, AT SHAFT NO. 4.

FIG. 2.—AIR-LIFT, DISCHARGING 1 200 GAL. PER MIN.

at $0.30 per 1 000 000 ft-gal., and the emergency pumping plant was also paid for under a lump sum, bid price. The water was measured by a 6-in. square orifice, a record of the head being obtained by a Friez automatic water register. The orifice was calibrated and checked from time to time by a box-weir and hook-gauge. The measuring tank and orifice are shown by Fig. 1, Plate XXXI, and all inflows, unless otherwise stated, are actual registered quantities.

The Rondout Pressure Tunnel is being constructed by contract for the City of New York under the direction of the Board of Water Supply, J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer; Robert Ridgway, M. Am. Soc. C. E., Department Engineer, Northern Aqueduct Department; Lazarus White, Assoc. M. Am. Soc. C. E., Division Engineer, Esopus Division; and Mr. Bertrand H. Wait, Section Engineer, Sections Nos. 5 and 6. The contractors are the T. A. Gillespie Company, Mr. Robert Swan, Vice-President and General Manager, and Mr. R. J. Gillespie, General Superintendent.

The writer wishes to express his thanks to Mr. Wait, who was in actual charge of the work, and collected the data from which this paper was prepared.

DISCUSSION

Mr. Pratt. Mason D. Pratt, M. Am. Soc. C. E. (by letter).—Two of the difficulties mentioned by Mr. Hogan are of interest to the writer because they correspond exactly with conditions met in driving an intake tunnel, in 1906 and 1907, from the power plant of the Central Pennsylvania Traction Company, at Harrisburg, Pa., to the Susquehanna River, a distance of 1 100 ft. This tunnel was driven entirely through limestone formation, and was from 5 to 20 ft. below the river level. In sinking the midway shaft and driving the headings for 200 or 300 ft. each way from the shaft, H_2S gas was encountered in such quantities as to cause serious inconvenience to the men, and for a time artificial ventilation was necessary. The gas affected the eyes, and the men required frequent medical attention; occasionally their eyelids were swollen so much that their eyes were entirely closed for a day or two. The tunnel passed directly under the yard of the Central Iron and Steel Company, and near two blast furnaces, and the gas was attributed to the furnace-slag filling which was used extensively around the yard of the steel works. This explanation was rational and satisfactory, but Mr. Hogan's experience on the Rondout shaft, where similar conditions did not prevail, would make it appear that the gas might have come from other sources.

At several times large volumes of water were encountered, the workings were flooded, and the pumping difficulties became most discouraging. Although the central shaft was only 45 ft. deep, and the writer did not have the additional difficulties of high-head pumping, as at Rondout, yet shaft pumps were necessary, and the troubles were not overcome until reciprocating pumps were abandoned entirely and there had been installed three No. 4 Emerson pumps, which at times delivered from 1 500 to 2 000 gal. per min. Grouting of seams was resorted to; the most serious case occurred when one heading had advanced 400 ft. from the shaft, and the seam was successfully closed with grouting forced through a 2-in. line from the top of the shaft, with an ordinary hand-power grout-pump, saw-dust being mixed with the grout.

One other difficulty on this work, which nearly caused its abandonment, was a large inflow of hot water from Paxton Creek, under which the tunnel passed. A short distance down stream from the center line of the tunnel, the creek had been dammed, and the pool was used as a reservoir for condensing water in the Traction Company's plant and for cooling water at the blast furnaces. This point was passed during the summer, when the flow in the creek was practically nil, and the temperature of the water passing into the tunnel was, at times, as high as 126°, so that the men had what was probably their first experience with a Turkish bath. This difficulty was not over-

come until the opening from the creek was located and surrounded with a coffer-dam. Mr. Pratt.

CHARLES B. BUERGER, ASSOC. M. AM. SOC. C. E. (by letter).—This paper shows that, in this work, the greatest obstacles to progress were: the difficulty of keeping the unwatering pumps in satisfactory working condition, when operating against a head exceeding 200 ft., and the large number of pumps with their connections needed in the small space available. For instance, at a depth of 300 ft., for a delivery of 725 gal. per min., ten pumps were used, although, ordinarily, this quantity could be delivered by two of the larger Cameron sinkers. Mr. Buerger.

The objectionable pulsations in the discharge piping were probably the result, not of the higher pressures, but of the increased length of the discharge column, with the correspondingly increased inertia of the water in this pipe, and greater water-hammer. Pumps of the shaft-sinking type are usually fitted with very small air chambers. It would be an interesting experiment to fit a chamber of considerable capacity on the discharge of such a pump. Such an air chamber could be designed to occupy a very narrow width, and be long enough to give the required capacity. For severe conditions an additional chamber could be placed in the middle of the length of the discharge pipe.

Considering the reduction in capacity of both pumps and all their piping and connections, as found in actual service, and the inconvenience caused by the fact that they occupied so much space, the use of such air chambers would seem to have a valuable field; they have been used successfully in the penstocks for water-wheels working under extreme conditions of length of supply pipe and variable load; only a service trial with the system, however, could determine the facts.

L. WHITE, ASSOC. M. AM. SOC. C. E.—The author has presented the eventful history of Shaft No. 4 of the Rondout Siphon so fully that it is difficult to add to the paper, or to dissent from what he has stated, or enlarge upon his sound conclusions. Mr. White.

It is perhaps well to re-state that no structural weakness of the rocks at this point gave any trouble, the case being simply that of a few porous layers heavily charged with water and H_2S gas. The Binnewater sandstone has a strong resemblance to the sand rocks of the oil region, but, instead of yielding oil and gas in great quantities, it yields only sulphur water and gas, which is valueless, unless, indeed, some day it is bottled as a new brand of Hunyadi or Pluto water. In the speaker's opinion, the sulphur gas gave the most serious trouble, for besides making it hard for men to work in the shaft, it forced the water out under pressure and maintained the large and high flow encountered.

As one of the superintendents well stated, "It would be hard to find another spot yielding water so persistently as Shaft No. 4." One

Mr. White. of the lessons to be learned from this work is to place as little reliance as possible on vertical sinking pumps hung in a shaft, and to use freely horizontal pumps in rings or chambers, only utilizing sinkers to pump from the bottom of the shaft to the lowest chamber. One of the most experienced shaft men even used horizontal pumps on the bottom, slinging them to chains in order to raise and lower them by the main cable at the time of shooting.

As described by Mr. Hogan, the grouting was of great aid in sinking the shaft, and was very successful, particularly in connection with a heavy seam at first encountered. It is not easy to determine whether or not the grouting of the diamond drill holes could have cut off much water, but it seems to have been effective, for, when the side of the shaft at the large chamber was drilled, large quantities of water were encountered in the holes within a few feet, but previously the shaft itself did not seem to be particularly wet at that level.

If this had been a permanent shaft, it probably would have paid to place a concrete lining in the wet zone to cut off the water. This could have been readily done by grouting off the water back of the concrete lining, as was done on a faulted zone at Shaft No. 5, where 125 gal. per min. were cut off. In this way the pumpage could have been cut down to almost the quantity found at the bottom.

It is interesting to recall that the troublesome layers in the shaft also proved troublesome in the tunnel just north of the shaft; that they yielded nearly 2 000 gal. per min. at one time for a stretch of scarcely more than 100 ft.; and that the grouting, unlike that in the shaft, was not successful in cutting off any flow, the porous rock being so broken up by faulting that the cement failed to cut off the water.

Eventually, the tunnel dried out the shaft, so that the chamber pumps were removed to the bottom, at which time there were four separate discharge lines in the shaft—two of 10-in., and two of 12-in. Eight air-operated pumps, of the Cameron reciprocating type, discharged into them. Three of these had a capacity of 450 gal., three of 350 gal., one of 200 gal., and one of 330 gal., making a total air-pump capacity of 2 930 gal. per min. against a 500-ft. head. In addition to these pumps three 6-stage Worthington centrifugal pumps were installed later. These were electrically operated, and their capacities were 560, 600, and 700 gal., respectively, making a total of 4 790 gal. per min. against a 500-ft. head.

Very wisely, a concrete bulkhead was built, having an opening closed by a heavy wooden door. Various pipes for air, water, etc., were embedded in the bulkhead when the concrete was placed. When an explosion of the dynamite magazine at the top of the shaft wrecked the transformer house, putting the electrical pumps out of commission, the door was closed, shutting off the water in the heading, and the remaining leakage through the bulkhead was handled by the

air-pumps. This probably saved the shaft from flooding. The ground-water at Shaft No. 5 has been lowered about 500 ft., and the tunnel adjacent is making about 1 350 gal. per min. Mr. White.

The bad ground at Shaft No. 4 retarded the work, so that excavation at this point was delayed about a year. Even with this delay, the T. A. Gillespie Company expects to complete the whole contract ahead of time. This is due to the great energy shown at the beginning, when the work was pushed while Shaft No. 4 was delayed. Had it not been for the energy and resourcefulness displayed by the contractor, the work might still be far from finished, and, instead of telling how the job was done, the engineers might be looking for some one to tell them how to do it.

H. M. Hale, Jun. Am. Soc. C. E.—This paper is a valuable contribution to engineering literature on difficult shaft-sinking. It describes what is without doubt the most trying piece of work which the engineers of the Board of Water Supply have encountered, and it is a matter of congratulation that the resourcefulness of the contractor accounts, in a large measure, for the opportunity to discuss the history of this shaft. Mr. Hale.

Shaft-sinking pumps will pump, but, under high heads, discharge lines with flexible connections act like the weakest link in a chain. The author fully points out their weaknesses in that respect, and the lesson to be learned is embodied in the successful application of concrete rings and chambers where the duties of the sinkers can be transferred to horizontal pumps with rigid connections.

Mr. White's discussion touches on the general conditions met in the tunnel driven north from Shaft No. 4, in which the strata of porous rock corresponding to that in the chamber were struck, resulting in an increased flow in the tunnel and an absolute loss of water in the shaft. It is fortunate that the intimate relation between the chamber and the porous zone, penetrated later in the heading, existed, on account of its bearing on the tunnel driven from Shaft No. 3 southward through the Helderberg limestone. The maximum flow into the tunnel (2 000 gal. per min.) lowered the water level in the whole region more rapidly than would have been the case had the greatest amount of pumping been confined to the water flowing into the chamber alone.

In the tunnel driven south from Shaft No. 3, approximately 1 000 ft. were in a zone of overthrust faults and vertical fractures, extending from Station 587 to Station 597. In every case the fractures running diagonally across the tunnel showed water-worn surfaces and cavities which had been enlarged by solution at a time when the region was above sea level. The result was that these cavities, most of which extended far above the tunnel roof and to unknown distances in both directions, were completely filled with a "residual" clay, which contained absolutely no grit. In one wide seam there was found a bed

Mr. Hale. of clean sharp sand, with a small proportion of pebbles. The bed thinned out at the edges where it pierced the surrounding clay.

In the tunneling these seams dropped their contents into the heading, but only in one case was there any flow of water into the tunnel. The first clay pocket containing sand gave a flow of about 200 gal. per min. Before the subsequent caves had been opened up, the chamber at Shaft No. 4 ran dry, causing the flow in the tunnel to decrease to less than 100 gal. per min. for its whole length. No water accompanied the inflow of soft clay after the first encounter; therefore only a small proportion of clay in the seams had to be taken care of. The seams cleared themselves wholly only for that portion of their length covered by a little more than the width of the tunnel. Beyond this, they held the clay contained in them and gave very little trouble thereafter.

Some of these caves are of considerable size, the largest being about 5 ft. wide, 20 ft. high, and extending for about 30 or 35 ft. to one side of the tunnel.

With such conditions, therefore, the results might have been far worse had the ground-water not been lowered below the level of the tunnel by the large inflow of water in Tunnel No. 4 North, which, at the foot of Shaft No. 4, is about 145 ft. lower.

Mr. Dunham. H. F. Dunham, M. Am. Soc. C. E.—The speaker would like to know whether it was necessary for the water removed to be taken out through some of the compartments of the shaft in order that the contractor could collect the stated price per million gallons for its removal, and whether the contract was worded so that the water taken from a well near the shaft would have been paid for by the City? If so, a 10- or 12-in. hole on each side of the shaft, drilled far enough below the porous rock to permit the use of an air-lift, would have served a double purpose. If drilled in the common Western way for water wells, with wooden rods, instead of a rope, much exact information as to the character of the rock crevices and horizontal seams would have been available; with a rope it is difficult to form an idea of what occurs in deep drilling. The rods enable a driller, not only to detect any slight cavity, but to determine very closely the vertical depth, under the drill, of any soft material or of any open spaces.

A heavy outfit is required for such large holes, but they give ready information as to the quantity of water in the strata penetrated. A sand bucket, often 20 or 30 ft. in length, is used, and is operated very quickly from a large drum.

In this way a fair estimate of the rate at which the water flows in at different depths can be made and used in providing the necessary machinery for its removal. A 10-in. hole, costing perhaps $5 per ft., would establish a series of facts greater in number and many times more valuable than little holes could furnish.

Mr. Dunham.

In using the air-lift, the efficiency, it is stated, depended on the back pressure. It would be of interest to know whether a large hole drilled much below the porous rock to give back pressure could be fitted with a heavy-pressure air-lift which would deliver a large volume of water from the required depth. The contract price for the removal of the water—about 7 cents per horse-power-hour, not including friction—would provide for some extra waste of energy from one or two single installations. In other words, a large-volume unit may show a profit where several small ones would not. To grout the porous strata through such a well on one side of the proposed shaft, while a quantity of water was being taken from a well on the opposite side, would help to carry the grout where it was most needed.

The speaker recalls a paper or a discussion by some member of this Society relating to the use of grout under pressure, under water in foundations, and in the protection of bridge piers, where it was desired to change loose rip-rap into a continuous solid mass. Are the modern methods of grouting related, in origin or design, to those which have been described before the Society?

Mr. Thomson.

T. Kennard Thomson, M. Am. Soc. C. E.—This paper would be much more useful if Mr. Hogan would add a summary—a tabulation or diagrams—showing the actual time spent on each part of the work, per day and per cubic yard, for pumping, sheeting, excavation, etc. If this summary included the actual cost it would, of course, be even more useful; and if it was arranged to show what was done each day from start to completion, including the number and size of the pumps and the quantity of water pumped, many contractors would be prevented from making foolish bids, and thus taking the work away from more experienced men.

In grout it is often cheaper to use a large proportion of cement to sand, as it runs more freely and sets more quickly. Some twenty years ago the speaker rejected a carload of English Portland cement, because, when mixed very wet, it set so hard in five minutes that one could not sink a finger nail in it; after standing in the laboratory for several months, however, this same cement required 2 hours for setting, showing, apparently, that it had been barreled too soon. The speaker has often wondered why, if a quick-setting cement like this can be turned out accidentally, manufacturers do not produce such a cement for purposes like those mentioned by the author, where it is necessary to stop leaks. For ordinary purposes, of course, engineers will continue to use a slow-setting cement.

The concrete lining in the tunnel seems to be thin, and the speaker is informed that in some of the rock tunnels it is not proposed to use any lining, which seems dangerous, because the sulphurous and other gases which have been encountered are likely to pollute the water, and, where a thin shell exists, destroy the concrete. In fact, it seems dan-

Mr. Thomson. gerous to build aqueducts with very thin reinforced concrete shells, for, even apart from the danger of the pollution and the destruction of the concrete, it is known that the hardest stone can be worn away by the continual rush of water, and that concrete has not, by any means, the resistance for this force possessed by hard stone; therefore, where concrete is used for aqueducts, there should be plenty of it.

Mr. Wait. BERTRAND H. WAIT,* ESQ.—Mr. Hogan's description is very thorough, but a few remarks in regard to the details of grouting in Shaft No. 4 may be of interest.

It has been well established from this and previous work of the same character that flows in water-bearing rocks can be cut off by the grouting or cementation process, if the water comes from pores, cracks, or seams which furnish any considerable amount. In other words, in most cases, grout can be forced in where water comes out.

The one case on this work where grouting was not successful was in the North Tunnel driven from this shaft. The rock encountered was the same strata which had furnished water in the shaft. In the North Tunnel, however, it was in a faulted and crushed zone, and while a stretch of it would yield a large quantity of water, any one individual leak was generally very small. Grouting here was not successful, as the grout did not flow freely.

In Shaft No. 4, when the heaviest flow was encountered and the largest seams struck, the grouting was particularly difficult. Pilot holes had not been drilled in advance, and when water was struck, the shaft had been excavated at one point to within 18 in. of the seams, and this 18-in. cover was badly cracked. When grouting was first started, a great deal of the cement was lost; by working in stages, however, first placing several batches of grout and letting it stand before continuing, the fissures were gradually closed, and a cover was made which would withstand the heavy pressures required in order to force the cement back into the small seams. During this process some of the pipes in the end of the shaft, opposite the point where the grouting was being done, were left open in order to relieve the pressure on the bottom.

One of the lessons learned from this was to keep the pilot holes always well in advance of cut holes when working in water-bearing ground. If there is a good cover which will withstand back pressure, the troubles of grouting will be lessened and a much better job will result. If the cover is leaky or is not strong enough to stand heavy pressures, low pressures should be used and vents should be left open, wherever possible, until the rock is grouted a few feet in advance. After this is set up, holes can be drilled through and heavy pressures used.

Mr. Wiggin. T. H. WIGGIN, ASSOC. M. AM. SOC. C. E.—The speaker, in the capacity of Designing Engineer, within whose province comes the

* Asst. Engr., Board of Water Supply, New York City.

design of pressure tunnels for the Board of Water Supply, can testify to the excellence of Mr. Hogan's description, as he made several visits to the shaft, and followed its progress with considerable care. Mr. Wiggin.

At about the time the engineers were having the greatest difficulty with this shaft, the speaker was sent to Michigan, by J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer of the Board of Water Supply, to investigate the comparative speeds and costs of driving vertical and inclined shafts and also long diamond-drill borings, and happened on a very wet shaft, being sunk by the Detroit Salt Company, in which the difficulties had been quite similar to, but even more pronounced in many respects than, those encountered in the shaft described by Mr. Hogan. This Detroit shaft, together with the Lieven Collieries shaft, attention to which had previously been drawn by J. F. Sanborn, Assoc. M. Am. Soc. C. E., formed a reassuring and suggestive background of successful experience with the grouting method.

Mr. E. F. Bradt, Mining Engineer, who directed the successful sinking of the Detroit shaft, has authorized the speaker to describe the methods used there, but the speaker finds that an article by Mr. Albert H. Fay, describing this shaft, has been recently published,* and he will merely call attention to a few facts of special interest.

Water was encountered in the Detroit shaft in three kinds of voids in the rock, namely, (1) relatively coarse voids in a thick bed of imperfectly cemented conglomerate or breccia at a depth of about 160 ft.; (2) horizontal seams, the largest of which, encountered at a depth of about 190 ft., was open and about 1 ft. thick; and (3) multitudinous pores appearing uniformly in a 10-ft. bed of sandstone, at a depth of about 410 ft., the pores being so fine that grout could not be forced into them.

The conglomerate was cemented by drilling into it a number of holes through gated nipples from the more impervious stratum just above it, and by forcing grout into the voids.

The seams, the largest of which, if broken into unexpectedly, would have let water in at an almost unlimited rate, were also filled through holes drilled from the stratum above; and the filling in of the largest seam was reinforced by grout pumped through several diamond borings, drilled from the surface, outside the shaft.

The porous sandstone was dammed off from the shaft by a solid diaphragm of grout, placed in a space formed by drilling holes about 9 in. apart through gated nipples, and blasting between them. This space extended out from the sides of the shaft and down on a slope of about 3 on 1 for a depth sufficient to cut off the 10-ft. bed. After sinking through this porous stretch, a reinforced concrete lining, with structural steel bracing, was placed within it as a safeguard and extra

* *Engineering and Mining Journal*, March 18th, 1911.

Mr. Wiggin.

water-stop, and this lining was extended through porous but less troublesome strata for a total depth of about 70 ft.

Hydrogen sulphide gas was so dense in the Detroit shaft as to require 2-hour shifts, even with the very liberal ventilation provided. Ventilation was furnished by forcing air at the rate of 18 000 cu. ft. per min. down the shaft through a heavy duck flue, 4½ ft. in diameter. The extreme necessity for this ventilation was sadly demonstrated by the death of several men, due to disconnecting the flue for a few minutes. The duck flue was very convenient in the crowded pipe-way, as it could be pushed in locally to permit increased working room at any point.

The noteworthy point in Mr. Bradt's method was the scrupulous care taken to keep in advance a pilot hole drilled through a gated nipple driven tightly into the rock, and, on approaching water-bearing rock, to drill all holes through gated nipples until the stratum had been grouted. As a result of the relaxation of care in drilling one hole, water, at the rate of more than 600 gal. per min., flowed

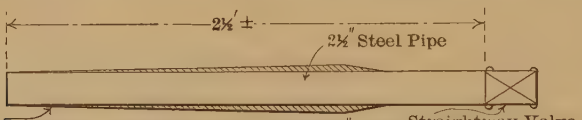

2½′ ±
2½″ Steel Pipe
Winding of "Shaker flannel" in strips 4″ ± wide.
Many feet tightly wound on. After winding,
thoroughly greased with tallow.
Straightway Valve
DETAIL OF PIPES THROUGH WHICH
DRILLING AND GROUTING WERE DONE.
DRIVEN INTO 2½″ HOLE BY ROCK DRILL.

FIG. 5.

into the shaft, filling it to the top, over which it ran at a rate of about 300 gal. per min. The shaft was recovered by pumping to within 45 ft. of the bottom and then sending a diver down to place a pipe in the hole, an operation which required only a few minutes. The impossibility of sinking this shaft without grouting in advance was aptly shown by Mr. Bradt's computation that the openings into the shaft, combined, would have discharged at the rate of 2 000 000 gal. per min., if supplied continuously with water at the observed head, which was 20 ft. above the top of the shaft.

Fig. 5 is a sketch of one of the nipples used at the top of the drill holes. This sketch was made from a sample seen at the shaft, and differs somewhat from the conventional illustration in Mr. Fay's article. To insert this nipple, a 2½-in. hole was first drilled to a depth of about 1½ ft. A driving cap was placed on top of the nipple, which was then driven by a regular rock drill. It was found that pipes driven in this way never came loose. In a test with a differential block a weight of 20 tons was required to pull one of these nipples.

Mr. Wiggin.

The shaft at the Lieven Collieries was apparently an example of ingenious extrication (not only once, but twice, in sinking the shaft) from a predicament in which the constructors would not have found themselves if they had used the methods developed at the Detroit shaft and later at the Rondout shaft. The excavation in the Lieven shaft was pursued until the water threatened to drown out the pumps, the flow being about 1 200 gal. per min.; then the bottom was apparently too porous to retain grout forced into drill holes, and an expensive sectional cast-steel inverted dome was placed in the bottom, and grouting through holes into the rock was done under this dome. Very great credit is due for this pioneer work in the use of grout for wet shafts, but the methods previously described are obviously a long step in advance in the application.

With respect to the troubles which occurred in keeping the sinking pumps in the Rondout shaft anchored and connected with sufficient secureness, and the apparent impossibility of running them at their rated capacity, it is pertinent to call attention to the more successful experience at a very wet shaft at Tombstone, Ariz., with some 14 by 8 by 12-in. duplex pumps, manufactured by the F. M. Prescott Steam Pump Company. The sinking of this shaft has been described by Mr. Elton W. Walker.* The flow of water in the shaft averaged about 2 400 gal. per min. Four of these sinking pumps were placed, two in each of two 5 ft. 9 in. by 7-ft. compartments. The pumps had a rated capacity of 600 gal. per min. against a 250-ft. head, and gave excellent satisfaction, two of them remaining in continuous operation at the bottom of the shaft for several years without overhauling. The pumps were submerged several times, once to a depth of 90 ft.; in order that they might be kept in operation under these conditions, all nuts which were likely to work loose were secured with cotter-pins. The pumps were handled by 5-ton, duplex, chain blocks, which ordinarily took most of the weight, although iron straps were provided to support the pumps while the blocks were being moved. The pumps were braced by jacks re-acting against the timbering, these jacks being hung from ropes to prevent them from dropping if they became loosened. The pumps were lowered 5 ft. at a time, that being the distance between the timber sets, and an expansion joint with a movement of 5 ft. was provided with each. Many other details relating to the pumping equipment at the Tombstone shaft are worthy of study.

Mr. Hewes.

V. H. Hewes, M. Am. Soc. C. E.—This exceedingly interesting and instructive paper shows the necessity for careful preliminary study in laying out a scheme of handling difficult shaft-sinking and tunnel work. The engineers deserve a great deal of credit for overcoming successfully the troublesome conditions which arose. Every piece of mining work requires special handling, due to its location relative to the

* "Sinking a Wet Shaft at Tombstone," *Mining and Scientific Press*, February 20th, 1909.

Mr. Hewes.

source of supplies, the local geological formation, and the object to be accomplished. After the general scheme has been formulated and the work is in progress, new difficulties arise, and the engineer is required to use his ingenuity, and knowledge based on his own experience as well as on what others have done, in solving his problems.

Several years ago, the speaker was employed in sinking a wet shaft in mining work, some details of which may be of interest. The shaft was in three compartments, two being for hoisting and the third for pipe lines and manway, having a landing at every third set. Each compartment was 4½ ft. square in the clear. The timbering consisted of "sets" of 10 by 10-in. wall-plates, back of which 2-in. lagging extended from one "set" to the next; the "sets" being 6 ft. apart on centers. The vertical corner posts were 10 by 10-in. sticks, gained ½ in. into the wall-plates, the latter being halved at the ends. All the timbers and lagging were wedged tightly in place, cord-wood backing being used where necessary. At intervals, "hitches" were cut in the rock, and extra long wall-plates were used to hold the timbers above. In "sinking," the lower three "sets" were tied together with temporary suspension rods, about 7 ft. 4 in. long, to prevent the timbers from falling to the bottom when loosened by the shots, and also to facilitate lining up.

Most of the water was from near the surface, and, to take it up, a short drift was run from the shaft, and in this was placed a sump and pump. A dam was built around the shaft back of the lagging, and the water was conducted to the sump, where, together with all water from the sinking pump, it was raised to the surface by a station pump.

A large sinking bucket was used, a guide-frame preventing the bucket from catching on the shaft timbers. This frame consisted of a horizontal piece of timber, with rounded ends to prevent it from catching in the guides, while iron plates at the ends extended on each side to hold it in place. An **A**-frame of rods, with hardwood blocks fastened to the apex of the frame, was attached to the horizontal timber at its ends. When the sliding framework reached the end of the guides, from 18 to 24 ft. above the men working in the bottom of the shaft, angle irons fastened to the ends of the guides caught the frame, the bucket continued on its way below the timbers, while the rope ran through the holes in the hardwood blocks.

In sinking below a working level, a cage carrying a car was used, the cage having extended guides braced with ties and struts, and forming a sort of truss. Thus the guides on the cage never left the shaft guides, although the platform of the cage carrying the car was below the timbering. When shooting the holes in the bottom of the shaft, heavy lagging was placed across the top of the bottom wall-plates, in order to prevent pieces of rock from being thrown up into the timbering and falling on the men. This lagging was held in place

by planks across the ends, a post being wedged between the plank and the bottom of the wall-plate above. The men would remain on the surface for an hour or more after shooting. This lost time was overcome by placing in the shaft a rectangular wooden flue, about 6 by 6 in., inside, and extending it to the bottom of the shaft, where a telescopic joint was placed to drop it below the lagging after shooting. At the hoisting works floor, a pipe was run from the air receiver into this flue and turned up with a small nozzle at its end. As soon as a shot was fired, the air was turned on, forming a suction in the flue, the men immediately descended, took up two pieces of lagging, dropped the telescopic flue below the lagging, and at once began at the opposite end to remove all lagging and proceed with their work. After reaching a working level, the flue was extended into the levels with equally good effect. Mr. Hewes.

An incident showing the necessity for ingenuity might be mentioned. At a location 500 miles from a railroad and 45 miles to the nearest mine having a lathe, the sinking pump failed. The water was about to drown the pump, when it was hoisted to the surface, where it was found that the piston rod was broken off at the piston. The miners were ordered out with their drills, and carpenters were set to work to build a small tank in the boiler-room and connect it by a sluice to an open box at the head of the shaft. A large sinking bucket was used to bail the water. A man stationed on the timbers at the bottom of the shaft tipped the bucket so that it would fill quickly, a man at the top of the shaft hooked a chain into a ring on the bottom of the bucket as it arrived at the surface, the other end of the chain being fastened to the cross-timbers of the gallows-frame overhead. The engine runner then lowered slowly; this tipped the bucket and emptied the water into the open box on the hoisting works floor, from which the necessary supply to run the works and saw-mill was conducted to the small tank in the boiler-room, from which it was pumped by the boiler feed pump into an elevated storage tank back of the works. The only changes required at the feed pump were the placing of a valve in the suction pipe from the storage tank, another valve in the suction pipe leading to the temporary tank, and a short connection and valve between the discharge of the pump and its suction pipe leading to the elevated storage tank. Only one 8-hour shift was lost, the miners on the next shift going to work as usual. Five days later the repaired pump was placed, without any further loss of time.

There is one other matter which is of interest. When mine workings are allowed to fill up with water and are later pumped out, care must be taken in entering the unwatered workings, as heavy falls of roof invariably take place, due to relieving the pressure from the working side while the pressure in the rock body decreases much more slowly.

Mr. Hogan. JOHN P. HOGAN, JUN. AM. SOC. C. E. (by letter).—Mr. Thomson suggests that a summary be furnished showing the actual time spent on each part of the work, etc. It would seem that further details could be only of special interest, and would be studied by few. No records of detailed costs are available, as the work was done by contract.

Several brands of quick-setting cement are now on the market. "Lias lime," a quick-setting compound, was introduced into the United States by the contractors for the Long Island Tunnels of the Pennsylvania Railroad, and was also used on the Belmont Tunnels. These quick-setting compounds are very useful in stopping leaks in compressed-air tunnels, but, in grouting seams in rock, there is some danger that they may set too quickly, thus blocking the seams without filling them.

The concrete lining in the Rondout Pressure Tunnel has an average thickness of 17 in., and none of the tunnels, either grade or pressure, on the Catskill Aqueduct will be left unlined, chiefly on account of the resultant decrease in hydraulic properties. The lining in the pressure tunnels is also called on to withstand the ground-water pressure when the tunnel is empty and a grouting pressure of 300 lb. per sq. in. The concrete is very dense, a 1:2:4 mixture being used, and specimens stored in sulphur water for a considerable time show no deterioration in strength. When it is considered that the tunnel, when full, is subjected to a maximum, unbalanced, internal pressure of about 145 lb. per sq. in., and that any leakage will be outward instead of inward, there can be little fear of deterioration of either the concrete or the water due to the action of gases or mineralized waters. The velocity of the water in the tunnel will not exceed 6 ft. per sec., and, as it will be clear and free from sediment, it seems to the writer that there will be no danger of erosion.

Mr. Dunham's discussion is very much to the point, and is extremely interesting to the writer, as the suggestions made by him were considered during the progress of the work. When the 4-in. bore-hole and pumping experiments described in the paper were first contemplated, it was proposed to sink a 10-in. hole, but there was no apparatus at hand and no contract then in force provided for a hole larger than 4-in. As the time for preparatory work was limited, it was reluctantly decided to rely on the data which would be furnished by the smaller hole.

With regard to the use of one or more 10-in. or 12-in. holes outside the shaft for pumping the wells, it may be stated that this method has been used in the Pennsylvania coalfields with excellent results, and there was nothing in the contract which would have prohibited payment for water pumped in this manner. There is a distinct advantage in having the pumping equipment separate from the shaft, as the chief

delays in sinking are due to the pumps and their handling. It would not be sufficient to have these holes extend through the porous rock; it would be necessary to carry them to the bottom of the shaft, or, if an air-lift were used, to about 250 ft. below the bottom, or a total depth for each hole of 750 ft. After due consideration, the contractor decided that the time and money required for sinking these holes would be better spent on the shaft itself. Mr. Hogan.

It is the writer's opinion that complete grouting could not have been successfully done through these holes, as one or both of them might have failed to hit some of the seams. For certainty in grouting, 4-in. bore-holes, 4 ft. apart, around the perimeter of the shaft would have been necessary. This scheme was also considered and abandoned on the ground of delay and expense. Either of these schemes might have been more economical in the long run than the methods used, if they had been undertaken at the start. When the shaft was down 250 ft., however, and the trouble was encountered, there seemed nothing better to do than to keep on as described in the paper.

Mr. Dunham is right in assuming that modern methods of grouting are related in origin and design to those used for a number of years previously. Grouting under pressure has been used for filling voids behind tunnel linings on the New Croton Aqueduct, on all the subaqueous tunnels in the vicinity of New York City, and at many other places. The ready adoption of grouting as a means of sinking this shaft was largely due to the fact that Robert Ridgway, M. Am. Soc. C. E., Department Engineer, and Robert A. Shailer, M. Am. Soc. C. E., Managing Engineer for the T. A. Gillespie Company, had both used grout extensively in tunnels at various times previous to this work.

While the idea of grouting under pressure to facilitate excavation in wet ground is old, the writer believes that its successful application is comparatively new. As far as he knows, the method was first applied successfully to shaft sinking as described in the *Engineering and Mining Journal* of August 18th, 1908.

Referring to Mr. Wiggin's discussion, it may be stated that after grouting had been decided on at Shaft No. 4, Mr. Bradt's experience in the Detroit Salt Company's shaft, as reported by Mr. Wiggin, increased confidence in the eventual success of this method, and some of the details worked out there were also of service on the Rondout shaft. The conditions in the Detroit Salt Company's shaft were undoubtedly much more severe than those in Shaft No. 4. Mr. Wiggin states:

"The impossibility of sinking this shaft without grouting in advance was aptly shown by Mr. Bradt's computation that the openings into the shaft, combined, would have discharged at the rate of 2 000 000 gal. per min., if supplied continuously with water at the observed head, which was 20 ft. above the top of the shaft."

Mr. Hogan.

While this statement may be true enough on the face of it, it seems to the writer to be highly misleading as the word, "if," presupposes two things: (1), an inexhaustable reservoir capable of supplying nearly 3 000 000 000 gal. of water per day (4 500 sec-ft.); and, (2), the absence of friction, which experience shows to be enormous in rock crevices. It need only be stated that application of the same methods of computation to the Rondout shaft would have indicated a possible flow from one seam alone of 500 000 gal. per min., whereas the total flow from the entire wet stretch, when encountered in the tunnel, never exceeded 2 000 gal per min.

Mr. Wiggin calls attention to the more successful handling of sinking pumps in a very wet shaft at Tombstone, Ariz. It should be noted that at this shaft, station chambers were placed every 200 ft., which accounts for the success of the sinking pumps and emphasizes the writer's statement that sinking pumps cannot be relied on for a head greater than 200 ft.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1202

SOME TESTS OF LARGE STEEL COLUMNS.*

By James E. Howard, Esq.

With Discussion by Messrs. Albert Lucius, Leon S. Moisseiff, A. W. Carpenter, J. S. Branne, Lewis D. Rights, J. R. Worcester, Horace E. Horton, Arthur N. Talbot, James Christie, N. R. McLure, R. S. Chew, George N. Cole, F. P. Shearwood, Charles Worthington, F. C. Kunz, and James E. Howard.

This paper describes the tests of five columns, representing members of Bridge No. 42, of the Pittsburg Division of the Pittsburg, Cleveland, Chicago, and St. Louis Railway, over the Ohio River, at Steubenville, Ohio. Through the courtesy of J. C. Bland, M. Am. Soc. C. E., Engineer of Bridges, Pennsylvania Lines West of Pittsburg, the writer is enabled to present the results of these tests.

The tests were made at Phœnixville, Pa., excepting the preliminary loading of Column No. 1, which was done at Watertown Arsenal. The tests were conducted by the writer jointly with Mr. C. P. Buchanan, Assistant to Mr. Bland.

The outside cross-section dimensions of the columns were approximately 20 by 30 in., each having a sectional area of about 90 sq. in. The length of one column was 20 ft., and that of each of the others was 36 ft. 5 in. For test purposes, they were provided with 10-in. pin ends; in the bridge, however, the connections were all riveted.

The type of the columns and the details of construction are shown on Fig. 1. The columns were made of 6 by 6 by $\frac{5}{8}$-in. angles, with outside web-plates 30 in. wide and $\frac{11}{16}$ in. thick; and, between the legs of the angles, which faced inward, there were web-plates $17\frac{1}{2}$ in. wide

* Presented at the meeting of April 19th, 1911.

and $\frac{5}{8}$ in. thick. The columns were single-latticed, with flat bars, $2\frac{1}{2}$ in. wide, $\frac{3}{8}$ in. thick, and 1 ft. $2\frac{1}{8}$ in. from center to center. At intervals there were diaphragms of $\frac{7}{16}$-in. plate. Steel rivets, $\frac{7}{8}$ in. in diameter, were used, and were driven in holes which were sub-punched $\frac{3}{16}$ in. and reamed $\frac{1}{16}$ in. larger than the nominal size of the rivets.

The ends were reinforced by pin-plates, the inside plates being $\frac{5}{8}$ in. thick and the outside ones $\frac{1}{2}$ in. thick, and each set was riveted to the web members with 39 rivets.

It will not be necessary to remark on the unusual opportunity afforded by these tests to acquire information on columns designed according to recent engineering practice, and in sizes representing work of magnitude. In addition to the determination of the ultimate strengths, observations were directed to matters of constructive details, and the behavior of the columns under loads preceding the final crippling ones.

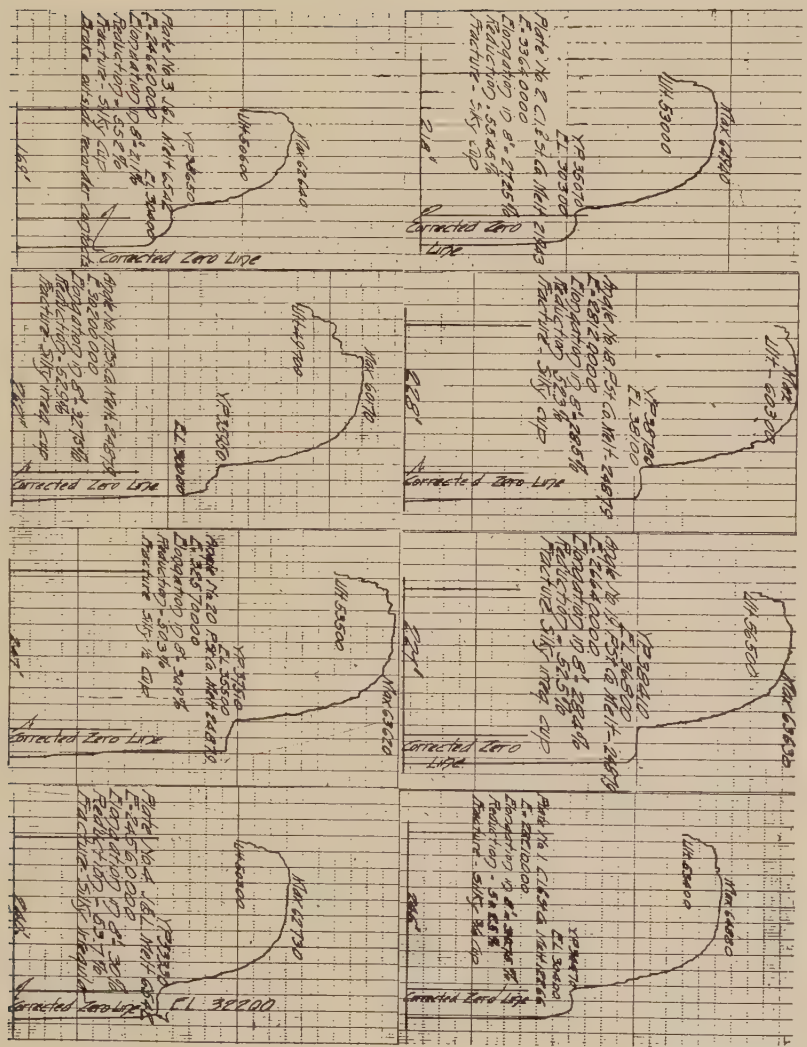
The general results of the tests are shown in Table 1.

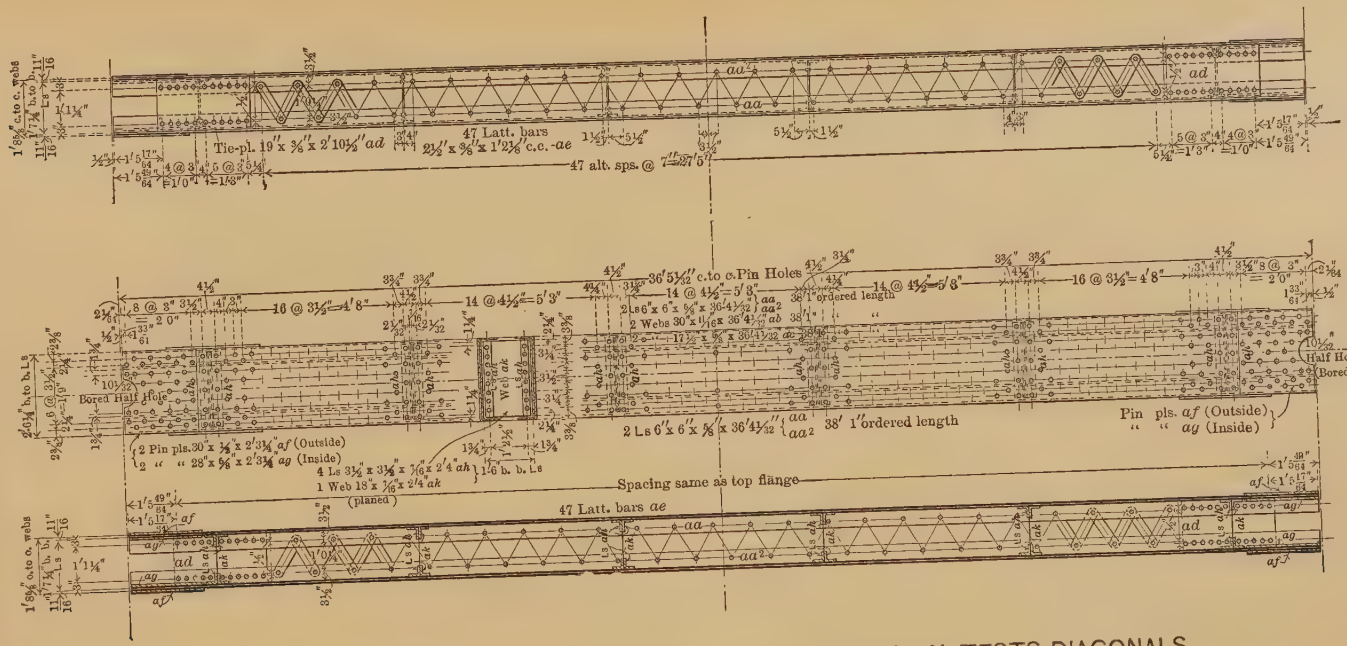
TABLE 1.—General Results of Tests of Bridge Columns.

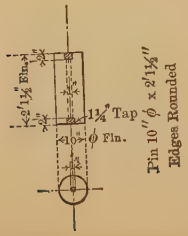
No. of column.	Length.			Sectional area, in square inches.	Maximum Loads Applied.		
	Feet.	Inches.	$\frac{l}{r}$		Gauge reading.	Total, in pounds.	Pounds per square inch.
1.....	20	0	26.2	90.73	806	2 600 962	28 667
2.....	36	5	47.2	90.33	806	2 600 962	28 794
3.....	36	5	47.1	90.78	829	2 675 183	29 469
4.....	36	5	47.2	90.32	845	2 726 815	30 191
5.....	36	5	47.1	89.96	850	2 742 950	30 490

The ultimate strengths of Columns Nos. 1 and 2 were not reached. Columns Nos. 3 and 4 failed locally at the pin-plates, immediately in front of the pins. Column No. 5 failed by the web-plates buckling inward, at a distance of 12 ft. 8 in. from the end, causing a sharp bend in the member at this place, with a sidewise deflection, parallel to the axes of the pins. In the direction of failure the radius of gyration was 9.15 in., against 9.84 in. at right angles thereto.

Although, in the tests of two columns, the maximum stresses which were capable of being sustained by the columns were not applied, while two others failed locally, it is believed, nevertheless, from the indications of the tests, that the ultimate strengths of all were fairly represented in that of Column No. 5.


Corrected Zero Line
Corrected Zero Line
Corrected Zero Line
Corrected Zero Line
Corrected Zero Line
Corrected Zero Line
EL 32200


Tie-pl. 19″ x ⅜″ x 2′10½″ ad
47 Latt. bars
2½″ x ⅜″ x 1′0⅛″ c.c.-ae
47 alt. sps. @ 7″=27′6″
16 @ 3½″=4′8″
14 @ 4½″=5′3″
36′5½″ c.to c. Pin Holes
2 Ls 6″ x 6″ x ⅝″ x 36′4½″
2 Webs 30″ x ½″ x 36′4½″
38′ 1″ ordered length
Bored Half Holes
Half Hole (Bored)
2 Pin pls. 30″ x ⅛″ x 2′3¾″ af (Outside)
2 " " 28″ x ⅝″ x 2′3¾″ ag (Inside)
4 Ls 3½″ x 3½″ x ½″ x 2′4″ ah
1 Web 18″ x ½″ x 2′4″ ak
(planed)
Pin pls. af (Outside)
" " ag (Inside)
Spacing same as top flange
47 Latt. bars ae



Rivets ⅞″ in diameter
All holes sub-punched 3/16″ and reamed

COMPRESSION TESTS-DIAGONALS

BRIDGE NO. 42 PITTSBURG DIV.

P., C., C. & ST. L. RY.

Required:

4 Test Diagonals
2 Test Pins

FIG. 1.

The grades of metal used in the fabrication of these columns are shown by Table 2, which gives the chemical analyses of the melts which furnished the plates and angles.

TABLE 2.—Chemical Analyses from Melts Furnishing Plates and Angles.

	Melt.	C.	Mn.	P.	S.
Plates	6 542	0.22	0.54	0.019	0.033
	16 266	0.27	0.51	0.007	0.034
	31 943	0.25	0.46	0.021	0.040
Angles	23 982	0.28	0.47	0.008	0.041
	24 879	0.18	0.53	0.007	0.032

Tests and analyses of rivet steel of the same grade as that used gave the results in Table 3.

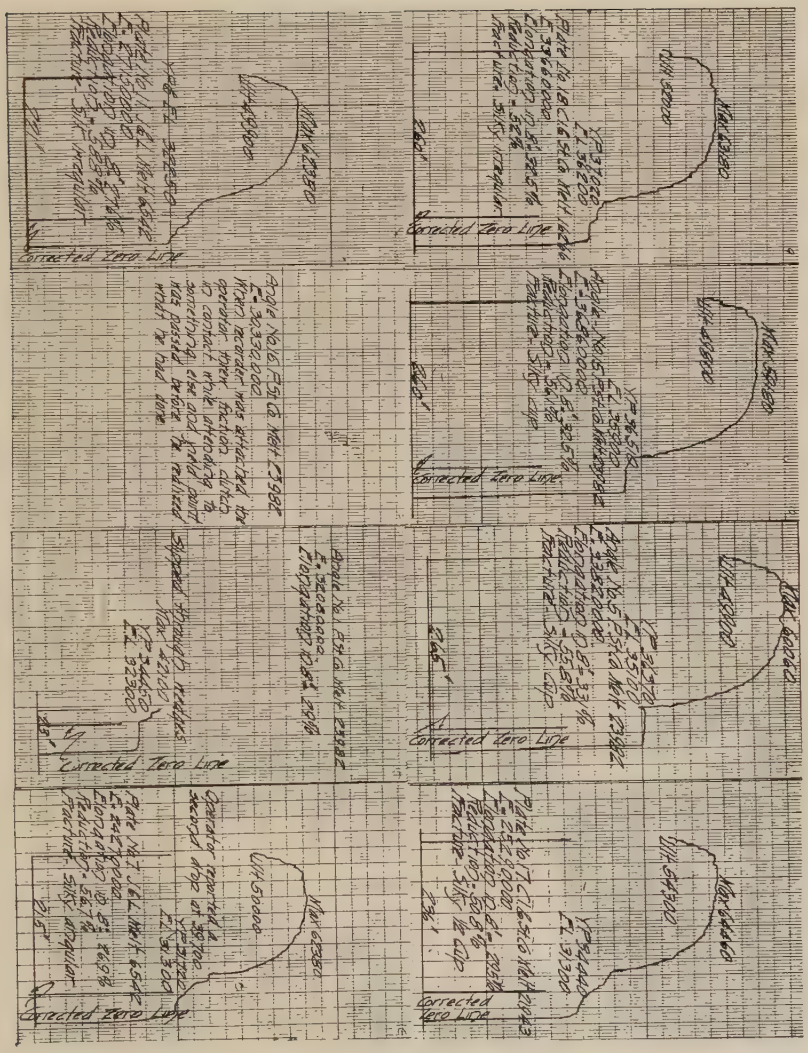
TABLE 3.—Results of Tests and Analyses of Rivet Steel.

Melt.	Pounds per Square Inch.		Elong. in 8 in.	Contraction of area.	C.	Mn.	P.	S.
	Elastic limit.	Tensile strength.						
8 174	35 120	52 580	33	64	0.09	0.45	0.007	0.033
12 218	38 430	53 510	33	68	0.09	0.35	0.007	0.029
16 283	33 730	51 370	34	65	0.09	0.51	0.007	0.030
18 184	36 320	50 180	34	66	0.10	0.45	0.007	0.037

The quality of the metal of the plates and angles is further shown by the stress-strain diagrams, Plates XXXII to XXXVI, from the records of the Pittsburg Testing Laboratory. Tests pertaining to the same column are grouped on the same sheet.

Concerning the applicability of the results of tensile tests to the behavior of compression members, it may be remarked that the elastic limits in metal of this kind are generally coincident in value in tension and compression, and the characteristics of the stress-strain curves are similar for loads up to and in the vicinity of the elastic limits.

Considerable variation appears in the elastic limits of the tensile tests in metal from the same melt, and, furthermore, the plates and angles assembled in the same column were characterized by differences of this kind. That a column would be benefited by having the elastic limits of the component parts of substantially the same value, there



COMPRESSION TESTS, NO. 2.

can hardly be a doubt, but, practically, a variation of some degree is to be expected.

Maximum and minimum values of the elastic limit and yield point found in the same melt are given in Table 4; and Table 5 shows the elastic limits of the principal component parts of each of the five columns.

TABLE 4.—Maximum and Minimum Values of Elastic Limit and Yield Point in the Same Melt.

Plate Metal.

Melt.	Elastic Limits, in Pounds per Square Inch.			Yield Points, in Pounds per Square Inch.		
	Maximum.	Minimum.	Difference.	Maximum.	Minimum.	Difference.
6 542	32 250	28 370	3 880	38 650	30 940	7 710
16 266	36 200	29 600	6 600	39 200	33 460	5 740
21 943	31 300	30 300	1 000	35 070	34 440	630
Metal of the Angles.						
24 879	38 100	30 000	8 100	38 780	35 950	2 830
23 983	37 300	30 200	7 100	38 810	32 860	5 950

TABLE 5.—Elastic Limits of the Metal Used in Each of the Columns.

No of column.	Elastic Limits, in Pounds per Square Inch.		No. of column.	Elastic Limits, in Pounds per Square Inch,	
	Plates.	Angles.		Plates.	Angles.
1	30 300*	38 100	4	33 200	34 500
	30 400	36 800		31 000	30 200*
	32 200	30 000*		28 370*	32 500
	30 400	35 500		31 000	32 600
2	36 200	35 900	5	34 600	36 900
	31 300*	35 100		29 600	33 800*
	32 250	32 300*		29 400*	35 060
	31 300	.....		31 300	37 300
3	32 700	31 800			
	35 100	31 500*			
	30 000*	34 500			
	30 700	31 400			

*Minimum values.

The maximum and minimum values of the elastic limit, as a rule, were not found in those specimens in which corresponding values of the yield point appeared. As an index of ultimate compressive resist-

ance, it is probable that the yield point is secondary in importance to the elastic limit, unless the two values coincide. It is believed that the minimum value of the elastic limit, as found in the component parts, chiefly modifies the ultimate resistance of the column, although it is probable that the shape of the stress-strain curve immediately beyond the elastic limit has an important influence.

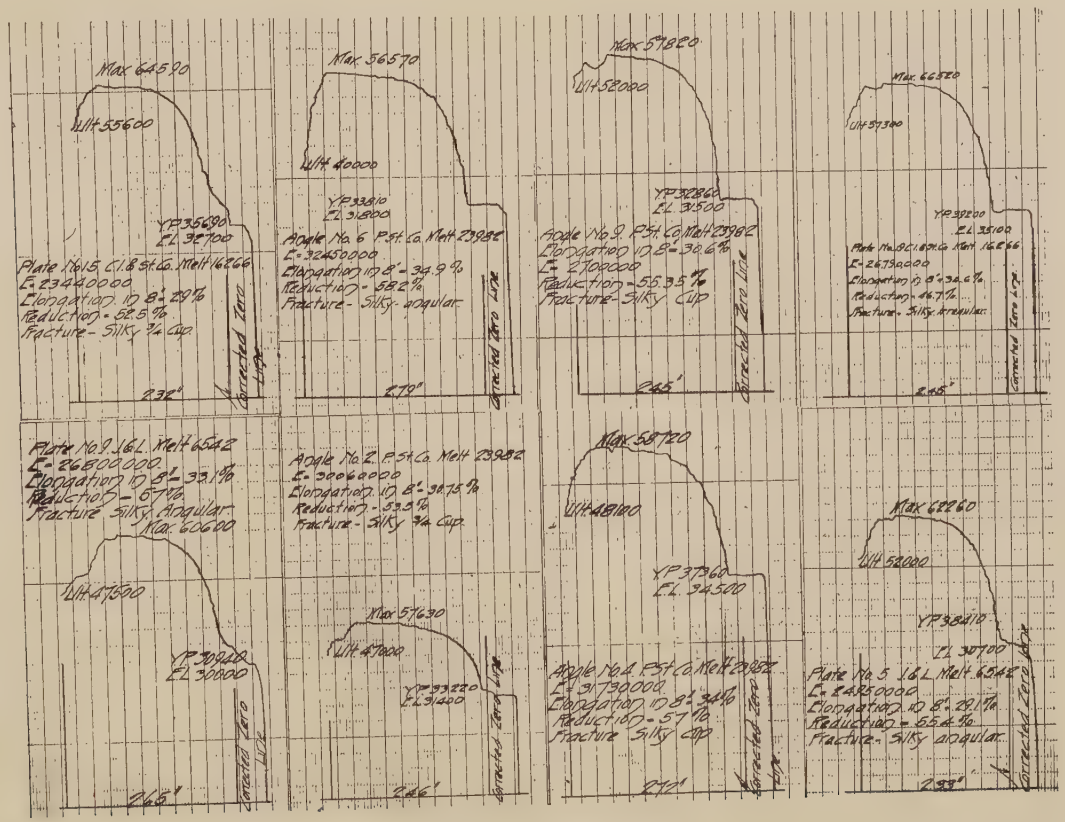
If it is accepted that the elastic limit has a controlling influence on the ultimate strength of a column (as it is believed to have), then variations of from 25 to 27% in this value, as witnessed in the tests of the metal of these plates and angles, would overshadow those considerations which find expression in empirical formulas for strength, and take no account of such features.

The ultimate resistance of Column No. 5 rose about 1 000 lb. per sq. in. above the elastic limit of the weakest plate, but did not reach that having the next higher value. The reinforcing effect of the higher elastic-limit-metal of the angles and that of some of the plates was not adequate to raise materially the strength of the column. While a compact type of column might receive substantial benefit from its more rigid members, it would appear from the present example that such would not be the result ordinarily. It cannot be asserted, by reason of lack of identity, that the weakest plate in this column actually was the one which failed first, but it seems reasonable to assume that it was.

The detailed information developed by the tests has been entered on a series of photographic prints, the entries being placed on those parts of the column to which they refer.

Fig. 1, Plate XXXVII, shows the longitudinal compression observed on gauged lengths established on different elements of Column No. 1. The compressive strains shown on this and subsequent prints were those due to a range in stress of 780 000 lb., between an initial load of 20 000 lb. and 800 000 lb., the maximum load applied at this stage. This range is equal to 8 597 lb. per sq. in., the maximum stress being equal to 8 817 lb. per sq. in. The maximum load, 800 000 lb., was applied several times before making the observations here recorded, which exhausted the early but slight permanent sets of the column, if there were any.

The compression of the web-plate of Column No. 1, on a gauged length of 150 in. established along the middle of its width, was



COMPRESSION TESTS, No. 3.

0.0408 in. At the edges of this plate the strains were 0.0394 and 0.0410 in., respectively, the mean of which is 0.0006 in. less than the observed strain on the middle gauged length.

Corresponding measurements on the edges of the web-plate on the opposite side of the column gave readings of 0.0467 and 0.0485 in., respectively.

The mean compression of the four corners was 0.0439 in., giving an apparent modulus of elasticity of 29 375 000 lb. per sq. in. This value is not far below that frequently assumed for steel, and therefore indicates a favorable action of the column taken as a whole.

The rate of compression on individual gauged lengths was nearly uniform, while the stress-strain curves for ascending and descending loads followed closely the same path. Table 6 shows the compressive strains which were measured on the center line of the web-plate, as the loads were successively advanced and released.

TABLE 6.—Compressive Strains.

Total applied loads, in pounds.	Strains under:		Difference.*
	Ascending loads.	Descending loads	
20 000	0.0003	0.0005	0.0002
100 000	0.0030	0.0032	0.0002
200 000	0.0078	0.0081	0.0003
300 000	0.0131	0.0137	0.0006
400 000	0.0187	0.0191	0.0004
500 000	0.0241	0.0246	0.0005
600 000	0.0296	0.0300	0.0004
700 000	0.0351	0.0355	0.0004
800 000	0.0410	.....	.....

* The difference represents a lag in resilience, in each case.

The compression of the column was measured along the center line of the web on two other gauged lengths, differing in their total lengths by 1 in., the longer one, 184.75 in., reaching from pin-plate to pin-plate, and the shorter length, 183.75 in., being taken wholly on the web-plate. The compressive strain from pin-plate to pin-plate was 0.0020 in. greater than that observed on the web. This strain, 0.0020 in., if it occurred on the solid metal, would correspond to a stress of about 60 000 lb. per sq. in., but the mean stress on the column was only 8 597 lb. per sq. in. In part, therefore, this strain signified an elastic movement of some other kind than a direct compression of the steel.

As the strains appeared and disappeared promptly when the loads were changed, it is inferred that there was no slipping of the pin-plates on the webs. None would be expected under as low a stress as that on the column at this time. The pin-plates were secured to the webs by the contractile force of 39 rivets, developing a probable gripping force of not less than $(0.6903 \times 39 \times 30\,000) = 807\,600$ lb. Considering that portion of the total load on the column which would be expected to come on the pin-plates, it would be thought that the riveting was fully adequate to prevent slipping, and that the compressive movement of 0.0020 in. represented combined compressive and shearing stresses, all of which were elastic.

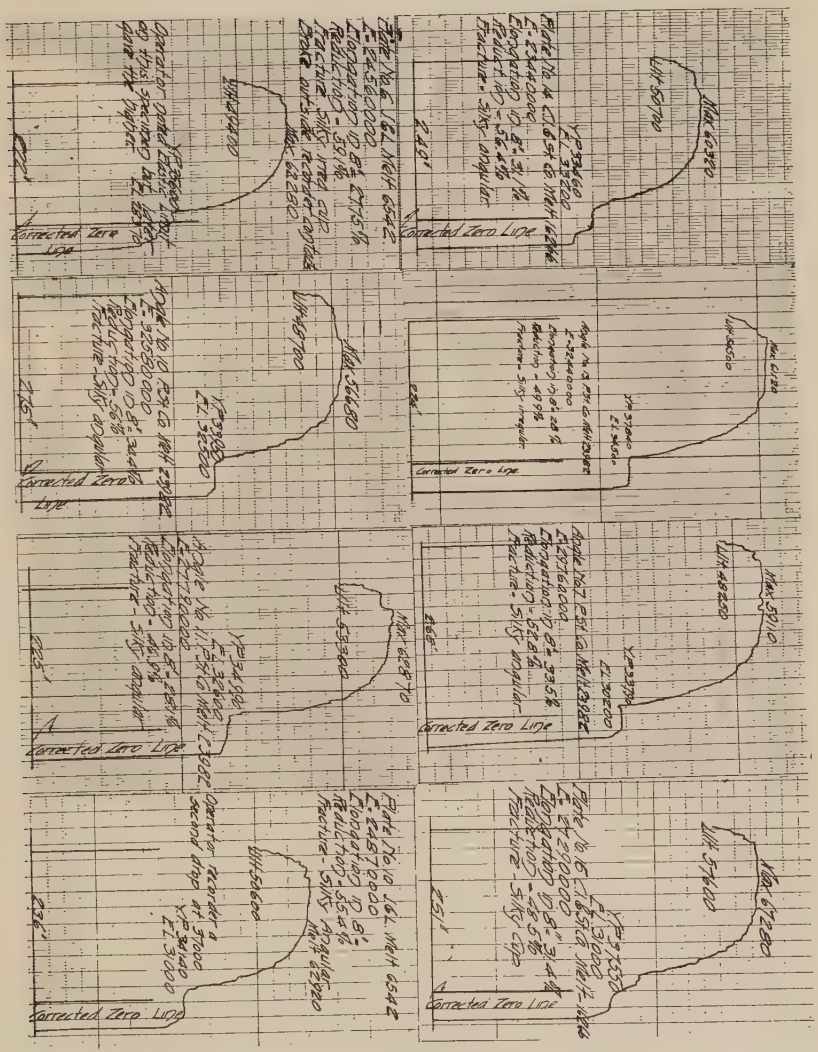
This result, however, illustrates the constructive difficulty encountered in any endeavor to attain a state of rigidity in the reinforcing plates comparable to that existing in the continuous web members.

The measurements which next followed were on gauged lengths of 10 in. each, referring to local strains in the column, over the same range in stresses as before. These results are entered on Figs. 2, 3, and 4, Plate XXXVII. On the north end, and also near the middle of the length of the column, five rows of gauged lengths were established along the center line, at the edges, and on intermediate lines, respectively. On and in the vicinity of the pin-plate at one end of the column twenty gauged lengths were established, as shown on Fig. 2, Plate XXXVII. At the opposite end of the column, observations were made on the center line only.

The range of stress applied called for a compressive strain of 0.0029 in., taking the modulus of elasticity at 29 500 000 lb. per sq. in. On the web the observed strains were found to range from 0.0026 to 0.0033 in. The influence of the diaphragm and connecting angles, in modifying the strains, if any, could not be traced in the results.

In stepping down from the pin-plates to the web, greater strains were observed than those on the web member. Here the strains ranged from 0.0033 to 0.0043 in., confirming the excess movements previously observed on the gauged length which extended from pin-plate to pin-plate.

The compressions of the pin-plates ranged from a plus strain of 0.0040 in. to a minus strain of 0.0006 in. Directly in front of the pin the compression naturally reached a maximum, while abreast this place,

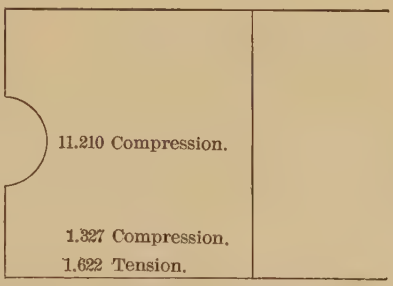


at the edges of the plate, the minus readings indicated that the metal there was in a state of tension. Diagonally inward from the pin, the metal was nearly in a neutral state, as regards resultant strains, the several readings being 0.0001, 0.0004, 0.0005, and 0.0003 in.

The compression on the bearing surface of the pin, when the load on the column was 800 000 lb., was 16 615 lb. per sq. in., using for a divisor the product of the diameter of the pin by the aggregate thickness of the metal of the web and the reinforcing plates.

For the purpose of having approximate figures in mind, which may serve in some degree as an index of the action of the pin-plate, it may be stated that the measured compression of the metal in front of the pin corresponded to a mean stress of $\frac{29\,500\,000 \times 0.0038}{10} = 11\,210$ lb. per sq. in., while the corresponding stresses at the intermediate gauged lengths and at the edges were 1 327 and 1 622 lb. per sq. in., respectively, the latter being a stress of tension.

The compression of the tie-plate shown on Fig. 1, Plate XXXVIII, was next measured. The longitudinal compression of this plate was that due to following and partaking of the compressive movement of the 6 by 6-in. angles, which in their turn followed the compression of the webs. It will be recognized that the efficiency of the riveting will be unfavorably studied in this plate, because of its attachment to a part of the column in which a complexity of conditions prevailed.



MEAN STRESSES, CORRESPONDING TO MEASURED STRAINS.
FIG. 2.

Measured near one of the lines of riveting, the compressions on three gauged lengths were 0.0008, 0.0012, and 0.0008 in. Checking these by one measurement over the aggregate length of 30 in., the reading obtained was 0.0030 in. The compression on the middle 10-in. gauged length is seen to be one and a half times that of the ends. This result is taken to signify that a progressive degree of compression occurs in the tie-plate, being a minimum at the end rivets and increasing with the number of rivets toward the middle of the length of the plate.

These results serve to direct attention to conditions which would be expected to prevail in a built-up member. The middle portion of a long plate, riveted to one of the principal members of the column, and measured on a longitudinal element, would reach a condition in which there would be uniform compression of the metal, practically. If the riveting did not reach to the end of the plate, some of the metal outside of but near the last rivet might be brought into a state of tension.

In a wide plate, the middle longitudinal element would be compressed less than the metal adjacent to the lines of rivets. This was exemplified in the present plate, where it was found that the compression along the middle of its width was 0.0011 in., against 0.0030 in. at the edge, both measured on a gauged length of 30 in.

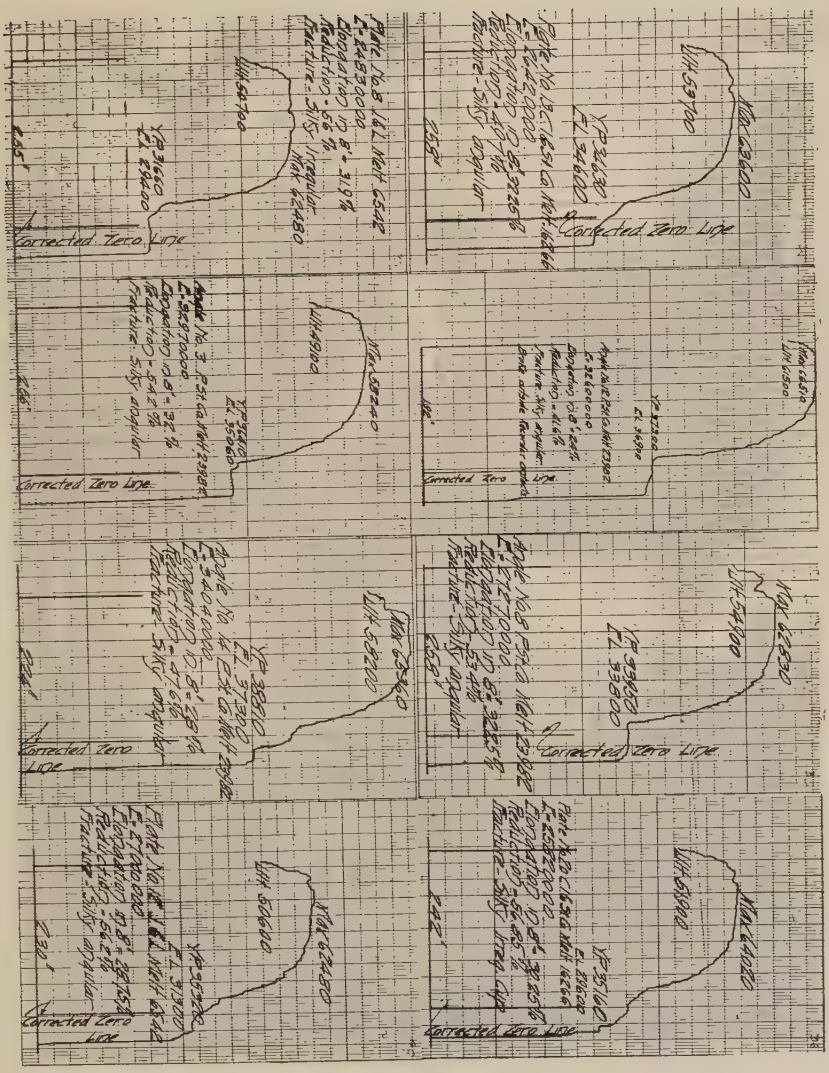
The presence of tie-plates at intervals along the length of a compression member would, necessarily, relieve the continuous member of a portion of the load locally, the intensity of the stress varying from section to section according to the disposition of these secondary parts.

The metal in the different parts of the cross-section of the angles probably carried its proportional share of the total load. The results of two determinations, as shown on Fig. 2, Plate XXXVIII, chanced to indicate a higher stress at the edge than that near the root of the angle. These results would have been reversed, one might think, if a substantial difference existed between the efficiency of the metal at the root and at the edges.

Observations were made on four of the lattice bars shown on Fig. 2, Plate XXXVIII, but in none of them were any strains detected, the range of stresses applied to the column being as first mentioned.

Some results on the lateral expansion of the column appear on Fig. 2, Plate XXXVIII, but detailed reference to them will be reserved until similar measurements entered on Figs. 3 and 4, Plate XXXVIII, and Fig. 1, Plate XXXIX, have been described.

The ratio of lateral expansion to direct compression has been found to have the value, $\frac{1}{3.55}$. Using this value, the predicted lateral expansion of the web-plate, 30 in. wide, based on the mean compression of the two edges, as entered on Fig. 1, Plate XXXVII, would be 0.0023 in. The observed lateral expansions ranged from 0.0019 to 0.0026 in. No modifications due to the transverse diaphragms were detected. On the pin-plates, notwithstanding the variations in direct compressive stresses which must have prevailed, the lateral expansions agreed closely, and


Corrected Zero Line
Corrected Zero Line
Corrected Zero Line
Corrected Zero Line
Corrected Zero Line
Corrected Zero Line

in one measurement coincided, with the predicted amount for the web. On the pin-plates, the high compressive stresses in front of the pins doubtless tended to modify the resultant lateral movements. This certainly was the case across the ends of the column, because each of those measurements showed minus lateral expansions of 0.0021 and 0.0025 in., respectively. These last readings, however, were consistent with the observations on direct compression, because the strains were in a tensile direction on certain of the 10-in. gauged lengths.

Turning back to the results entered on Fig. 2, Plate XXXVIII, three transverse gauged lengths were established in one plane, one of which had its extremities on the angles on the lines of the lattice-bar rivets, one near the inner edges of the angles, and one near the corners of the column at the roots of the angles. As it had been ascertained that no strains were developed in the lattice bars by the loads of compression applied to the column, it would follow that the lateral expansion on the intermediate gauged length, which had its extremities on the line of the riveting, would be expected to correspond to the increased height of the triangle made by two of the lattice bars and the section of the angle between them, the angle being compressed its proportional amount of the total shortening of the column. It was found that the lateral expansion, measured on the intermediate gauged length, 0.0012, corresponded closely to that called for by the above assumption, the difference amounting to only about 0.0001 in.

The lateral expansion of the metal of the angle itself should furnish a correction to apply to each of the other gauged lengths, in order to arrive at the expected readings. Here, again, a close correspondence was found between the expected and the observed results. The correction, subtracted for the inner gauged length and added for the outer one, called for results differing from the observed expansions only about 0.0001 in. in each case. In other words, the widening of the legs of the angles and the change in height of the triangle formed by the lattice bars and the included section of the 6 by 6-in. angle accounted for the observed results, as would be expected.

An observation was made, from angle to angle opposite the diaphragm, shown on Fig. 2, Plate XXXVIII. At this place the lateral expansion was 0.0001 in. as against 0.0012 in. on the corresponding length taken on the angles midway of the diaphragms. This difference in behavior was attributed to the restraining effect of the diaphragm.

This restraint might reasonably have led to the introduction of appreciable stresses in the lattice bars in that vicinity, but the connections were not sufficiently rigid, it would seem, for accomplishing that result.

When higher stresses were applied to the column, there was a slight compression of the lattice bar, shown on Fig. 2, Plate XXXIX, of about 0.0002 in. The column at this time was under a load of 28 667 lb. per sq. in. If the lattice bar was compressed the amount stated (some uncertainty pertained to this measurement), the corresponding stress was not more than 600 lb. per sq. in.

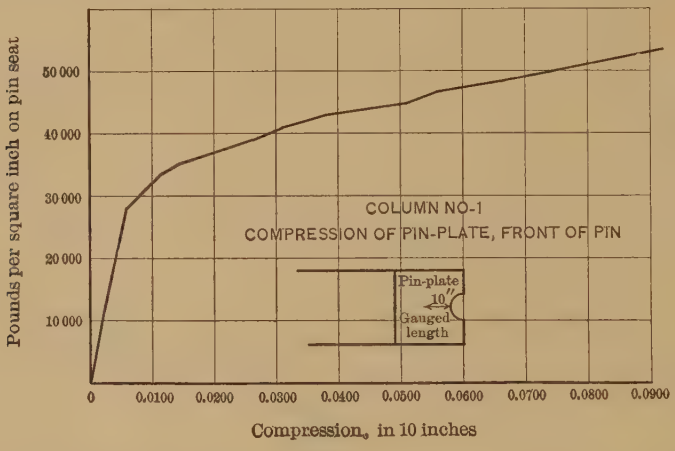


FIG. 3.

Further tests were made with this column, on the compression of the metal in front of one of the pins, using a gauged length of 10 in., as before. The results are plotted on Fig. 3. Rapid yielding occurred, as the load was increased from 28 000 to 33 000 lb. per sq. in., referring to the stress on the bearing surface of the pin. Although the stress on the sectional area of the column was then only from 15 000 to 18 000 lb. per sq. in., judging from its behavior in front of the pin, the ultimate resistance of the column could then have been predicted. It was apparent that the ultimate strengths of the columns would be found in the vicinity of 30 000 lb. per sq. in.

The measurements on this 10-in. gauged length, and subsequent ones on other columns of the series, were made with a strain gauge designed and used for the purpose of measuring strains in structural members under service conditions.

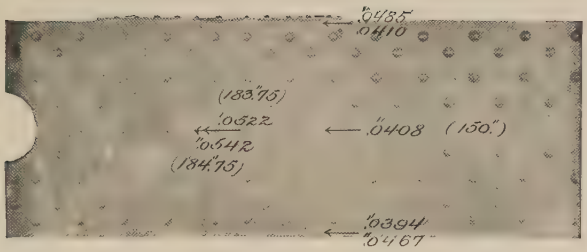


FIG. 1.—COMPRESSIVE STRAINS. COLUMN NO. 1, NORTH END. MEASURED ON GAUGED LENGTHS OF 150, 183.75, AND 184.75 IN.

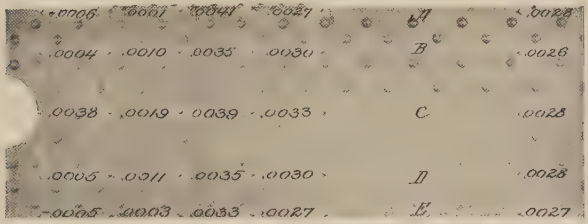


FIG. 2.—COMPRESSIVE STRAINS. COLUMN NO. 1, NORTH END. MEASURED ON GAUGED LENGTHS OF 10 IN. EACH.

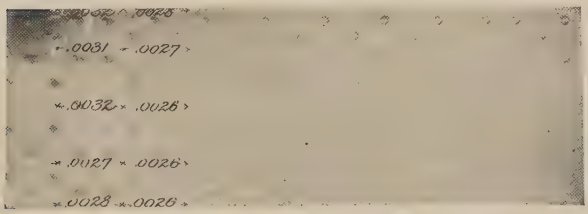


FIG. 3.—COMPRESSIVE STRAINS. COLUMN NO. 1, MIDDLE. MEASURED ON GAUGED LENGTHS OF 10 IN. EACH.

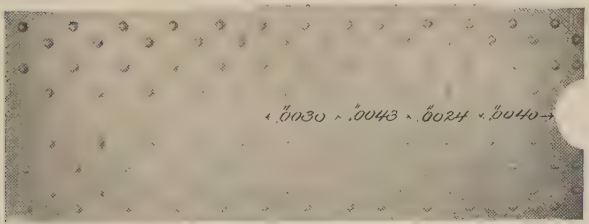


FIG. 4.—COMPRESSIVE STRAINS. COLUMN NO. 1, SOUTH END. MEASURED ON GAUGED LENGTHS OF 10 IN. EACH.

The disparity in sectional area and intensity of stress, between the body of the column and the pin bearings, directs attention and gives emphasis to a feature in this as well as in other compression tests, namely, that details of construction generally exert an influence on the behavior of the member, and not infrequently modify the ultimate resistance materially. Regularity of action is limited to stresses within, and ceases at, the elastic limit, and when over-straining of the metal occurs, locally or in general, the strength of the column is menaced.

The results next described refer to the permanent sets which were caused by the application of the maximum load to the column, and its release.

On Figs. 3 and 4, Plate XXXIX, are recorded the permanent sets, found on gauged lengths of 10 in. each, which resulted from applying a load of 28 794 lb. per sq. in. to Column No. 2. The compression on the bearing surfaces of the pins for this load was 53 353 lb. per sq. in. Necessarily, the greatest effect was produced in front of the pins. At one end of the column the permanent set was 0.1168 in., at the other end it was 0.1039 in. In other parts of the pin-plates the sets were comparatively of small magnitude, much less than found in the web-plates.

At the east end of the column the gauged lengths, which stepped down from the pin-plate to the web, showed the next largest movements to the metal directly in front of the pins. Evidently the pin-plate had moved bodily along the column in the direction of its length. At the opposite end the bodily movement of the pin-plate was not a pronounced feature of the test.

The mean set, per 10 in. of length, of the web of this column was 0.0030 in. The individual sets are seen to range from 0.0006 to 0.0180 in., showing an irregularity in the development of sets not unusual for stresses which are in the zone of, but just above, the elastic limit of the steel. In a built member the riveting may exaggerate the natural differences of the metal, and remarks in reference thereto will be made later in this paper.

The permanent sets of Column No. 3 were observed, on the center line, at each of the four corners, the results being entered on Figs. 1 and 2, Plate XL. The large set, measured on the north side and east end of the column, in stepping down from the pin-plate to the web,

was due to a dishing of the plates in front of the pin causing a bend in the web at the end of the reinforcing plate. The local bending had the opposite effect on the south side of the column at this end, where the slipping of the pin-plate appeared to be zero.

The earliest set of the metal of the pin-plate naturally occurs immediately at the front of the pin, thence extending forward as the loads are increased. The affected metal in these columns was generally confined within the limits of the first 10 in., measuring from the pin. Column No. 3, however, afforded an example in which the sets reached into the second gauged length. At the west end the yielding of the metal at the pin was more nearly the same on each side of the column than witnessed at the east end, where local failure at the pin-plate occurred.

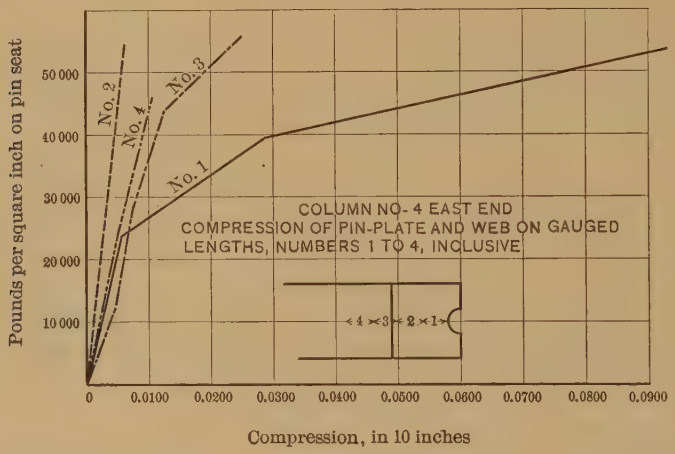


FIG. 4.

During the progress of the test of Column No. 4, the compressive strains developed on the reinforcing plates and the web, at each end of the column on the north side, were in succession measured by the strain gauge. These results are plotted on Figs. 4 and 5.

An inspection of the stress-strain curves of these diagrams shows the relative rigidity of the metal on each of the four gauged lengths. The metal least compressed was the part of the reinforcing plates covered by gauged lengths No. 2—a consistent result, it would seem.

Under the earlier loads, the greatest movements were observed on lengths No. 3, from which it appears that the reinforcing plates were carried along, lengthwise of the column, at an excessive rate when con-

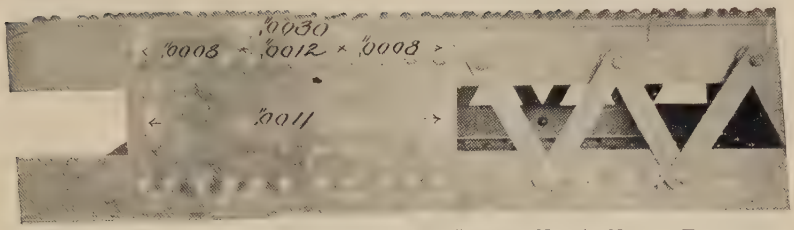


FIG. 1.—COMPRESSIVE STRAINS. COLUMN NO. 1, NORTH END.
MEASURED ON GAUGED LENGTHS OF 10 AND 30 IN., RESPECTIVELY.

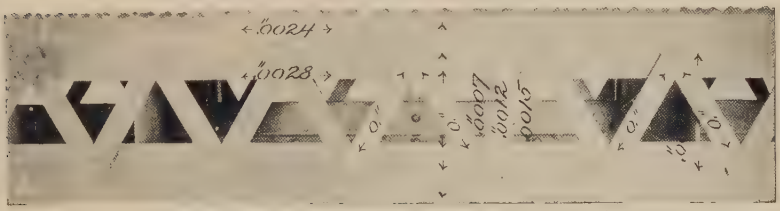


FIG. 2.—STRAIN MEASUREMENTS. COLUMN NO. 1, MIDDLE.
ON ANGLES, LATTICE BARS, AND ACROSS COLUMN.

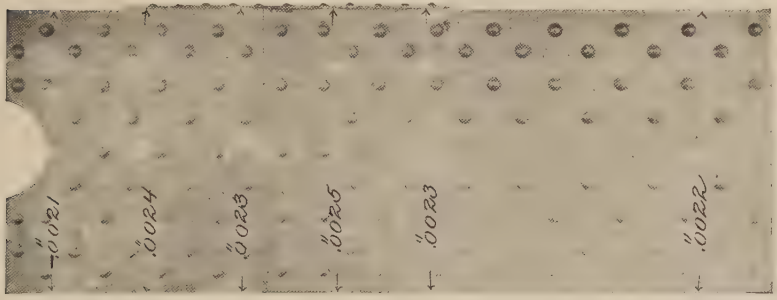


FIG. 3.—LATERAL EXPANSION MEASUREMENTS. COLUMN NO. 1, NORTH END.

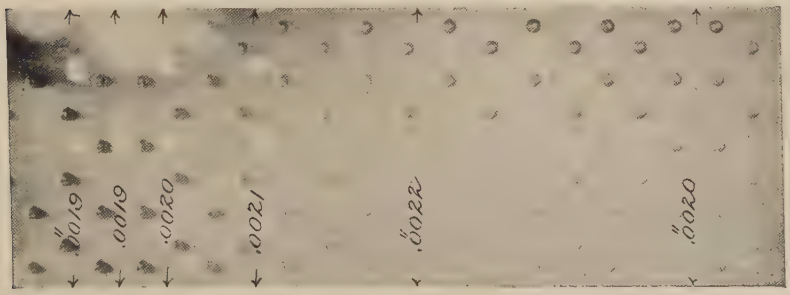


FIG. 4.—LATERAL EXPANSION MEASUREMENTS. COLUMN NO. 1, MIDDLE.

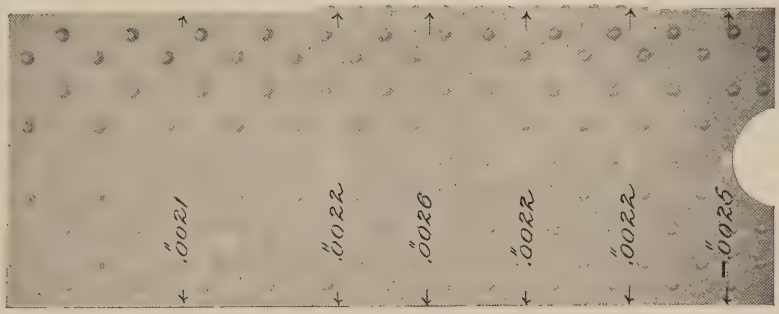


FIG. 1.—LATERAL EXPANSION MEASUREMENTS.
COLUMN NO. 1, SOUTH END.

FIG. 2.—STRAIN MEASUREMENT ON LATTICE BAR.
COLUMN NO. 1, EAST END.

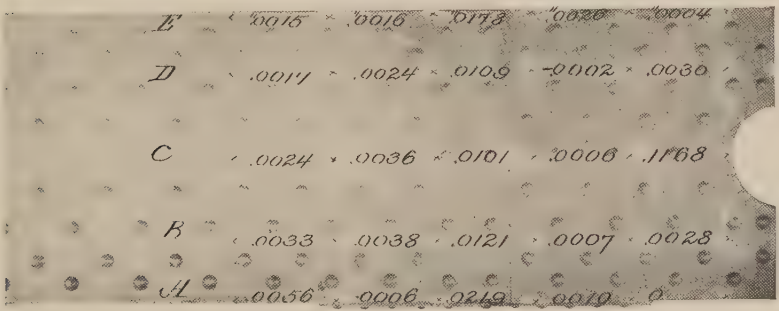


FIG. 3.—PERMANENT SETS. COLUMN NO. 2, EAST END.
ON GAUGED LENGTHS OF 10 IN. EACH.

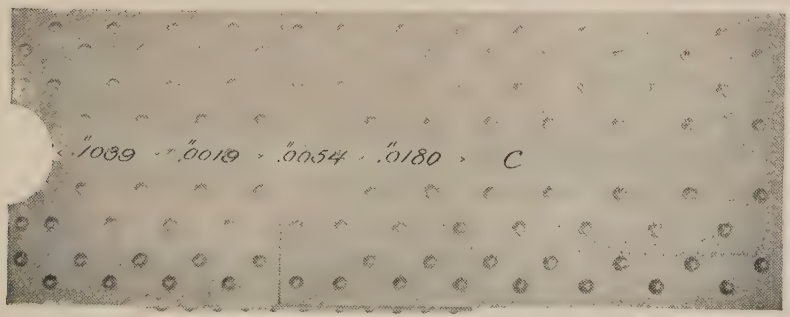


FIG. 4.—PERMANENT SETS. COLUMN NO. 2, WEST END.
ON GAUGED LENGTHS OF 10 IN. EACH.

sidering the movements observed on lengths No. 4. It is difficult to account for this excess movement, unless it is assumed that the pins were bent sensibly or that the riveting was inadequate to hold the plates firmly in place.

As the loads were advanced, the compressive strains directly in front of the pins soon gained the ascendency as regards magnitude, which of course was maintained for the remainder of the test, gauged lengths No. 3 standing next in the order of yielding.

Fig. 6 was prepared to show the measured compression of certain of the columns with reference to the compression pertaining to a modulus of elasticity of 29 500 000 lb. per sq. in. The stress-strain curves of

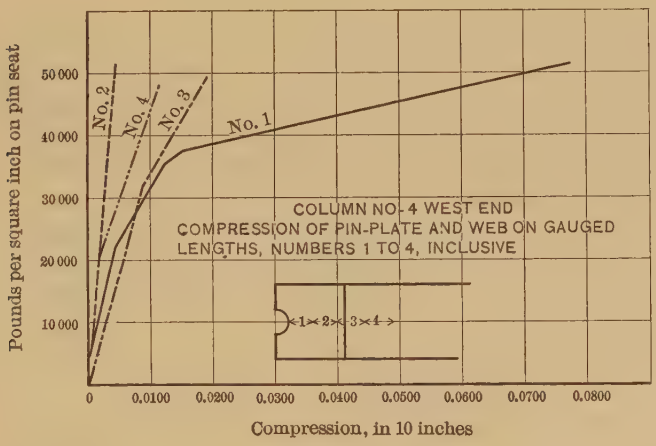


Fig. 5.

the columns are seen to follow closely the lines representing the above modulus, up to loads ranging from 12 000 to 20 000 lb.; the differences, if any, not showing on the scale used in the diagram. Under the higher loads, there is a gradual divergence, flattening the curve, and showing greater compressibility, which was probably due to the gradual development of permanent sets.

These curves represent the compressions on gauged lengths of 240 in. each, located on the web members. The extremities of the gauged lengths were nearly 100 in. from the ends of the columns, and, therefore, were quite beyond any local influence of the pins. The gradual development of sets seems to be characteristic of these built members, and is doubtless representative of columns of this type.

There are influences present in large built members which tend to modify their behavior, so that they do not act like plain steel bars. Plain bars of the grade of steel used in these columns would be expected to develop strains strictly proportional to the stresses in well-annealed, machine-finished specimens until the zone of the elastic limit was reached.

A jog in the compression stress-strain curve for loads just beyond the elastic limit would be looked for comparable to that displayed in the tensile tests. Such a jog sets a practical limit to the compressive strength of the material, excepting for very short pieces—for specimens

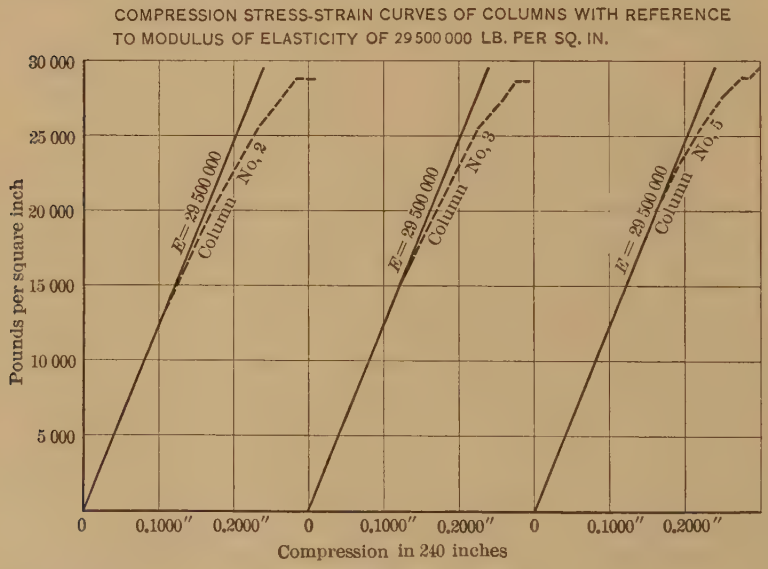


FIG. 6.

of the material rather than columns. In the case of a short compression specimen, frictional reinforcement, from contact with the platforms of the testing machine, raises the ultimate strength by reason of restricting lateral flow; and, provided there is symmetrical radial flow, no maximum resistance would be found. Such tests merge into those of cubic compression where, as far as known, no physical change results from the application of the highest stresses yet applied, and they have exceeded 100 000 lb. per sq. in.

It is known that, in steel and iron shapes, the parts first to cool after hot working are left finally in a state of initial compression. In

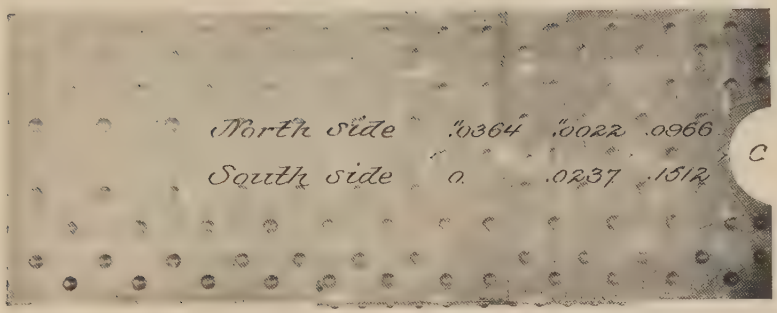


FIG. 1.—PERMANENT SETS. COLUMN NO. 3, EAST END.
ON GAUGED LENGTHS OF 10 IN. EACH.

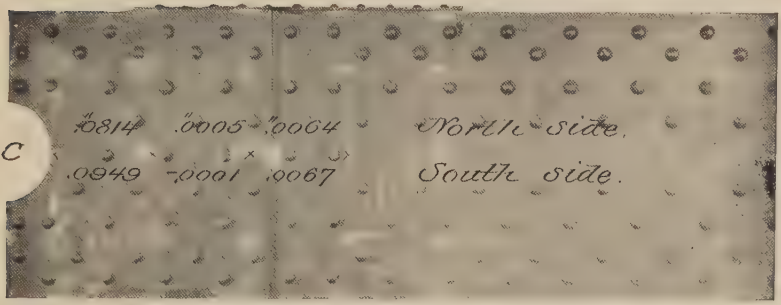


FIG. 2.—PERMANENT SETS. COLUMN NO. 3, WEST END.
ON GAUGED LENGTHS OF 10 IN. EACH.

FIG. 3.—COLUMN NO. 3 AFTER TESTING. SOUTH SIDE, EAST END.

FIG. 4.—COLUMN NO. 3 AFTER TESTING. EAST END.

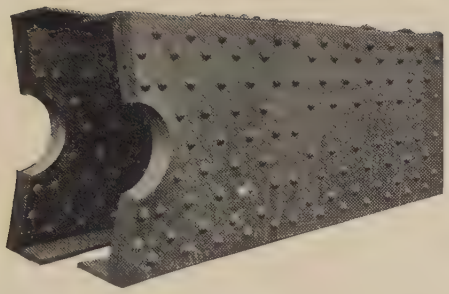


FIG. 1.—COLUMN NO. 4 AFTER TESTING.
SOUTH SIDE, WEST END.

FIG. 2.—COLUMN NO. 4
AFTER TESTING.
WEST END.

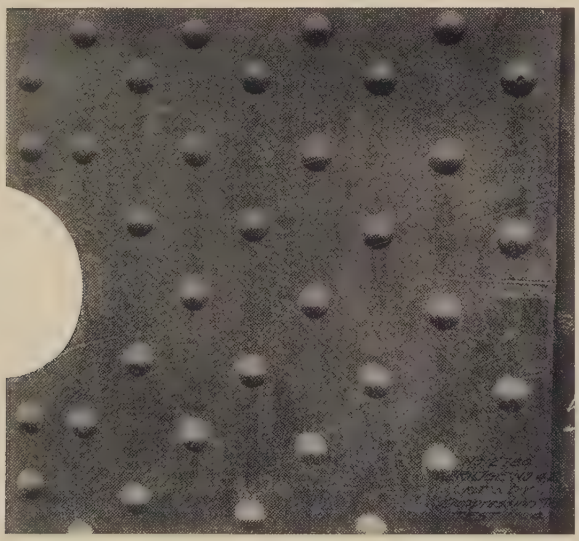


FIG. 3.—COLUMN NO. 4 AFTER TESTING.
NORTH SIDE, EAST END.

FIG. 4.—COLUMN NO. 5 AFTER TESTING.
SOUTH SIDE, AT PLACE OF FAILURE.

whatever direction the metal is tested the presence of initial strains tends to cause the progressive development of permanent sets, because of local over-straining of the metal under the combined effect of external forces and internal strains. The component parts of these columns were doubtless influenced by initial strains in some degree, which would cause the elastic limits to be vague and not sharply defined. Permanent sets, under these conditions, might be caused by stresses within the working loads, but confined to so small a part of the volume of the member as to render their presence negligible. It is, indeed, a matter of doubt whether any work of magnitude, involving the use of structural members of the kind under consideration, can be cited as an example in which no part thereof is strained beyond the elastic limit of the material.

The mean unit stress could be well within the elastic limit of the member, and yet local zones of overstrained metal might exist.

Initial strains, induced by the rapid cooling of the shapes from the temperature of rolling, may be augmented by initial strains introduced by mechanical operations on the cold metal. Straightening the shapes when cold, an essential operation, it would seem, introduces internal strains, and, unless the pieces are subsequently annealed, places them in a state of critical equilibrium, because of which they respond to applied stresses in part elastically and in part by the development of permanent sets.

Punching and shearing affect the metal over a wide area, as witnessed in the lines of scale which are disturbed about a punched hole or along the edge of a sheared sheet. The disturbance of the scale is evidence of the presence of internal strains. Reaming removes the most intensely strained metal at the immediate sides of the holes, but commonly does not remove all that was disturbed during the operation of punching.

At occasional intervals, internal strains may be introduced by the use of the drift-pin. When it is considered that the drifting of a 1-in. hole much beyond $\frac{1}{1000}$ in. over-strains the metal and introduces initial strains, it may be seen that any use whatever of the drift-pin is likely to be attended with the introduction of initial strains. The result may not be objectionable in a practical way necessarily, but such treatment contributes toward causing that divergence of the stress-strain curve in its upper portion from the right line representing a metal of no sets.

The contraction of the rivets in cooling, after driving in the direction of their length, is also a cause for the introduction of internal strains. Rivets, in cooling down over a range of from 900 to 1 000° Fahr., contract some five or six times the distance which will extend them to their elastic limits when cold. Provided the plates which are being riveted are nearly flat, and lie close together, the contraction of the rivets should exert a pressure approximating the elastic limit of the rivet metal.

Assuming, however, that the contractile force is only 30 000 lb. per sq. in. on the stems of the rivets, the aggregate gripping force reaches a very formidable amount in a column of this type. On one of the longer columns of this series, the total gripping force of the rivets through the web-plates and lattice bars, on the above basis, reaches a total of more than 20 000 000 lb.

The section of the column within the limits of the gauged length of 240 in. included 456 rivets driven through the web-plates. The estimated contractile force on this part of the column exceeds 9 000 000 lb. The ratio of the annulus of the rivet head to the area of the stem is 1.35, from which it follows that a contractile force of 30 000 lb. per sq. in. on the stems would cause a mean compressive stress of 22 200 lb. per sq. in. under the heads.

From the above statement of the case, it would appear that the internal condition of the column is one characterized by the complexity of the strains which are present. The metal is not in a state of repose initially, that is, prior to receiving the loads of the testing machine, or such as would come on it when in its place as a structural member. The factors mentioned are believed to be adequate to account for the gradual flattening of the upper part of the stress-strain curves, as they were found and illustrated on Fig. 6.

Permanent yielding clearly took place for some time prior to reaching the maximum stress; and, logically, sets would be expected to appear first in localities where the internal strains augmented the effects of the loads applied in the testing machine. Scaling in the body of the column was first observed in the vicinity of the rivets, a very natural place, it would seem, for the metal to display the first signs of overstraining.

On a close approach to the ultimate resistance, time becomes a factor in the total amount of compression displayed by the column, the full compression not being reached immediately, as in the case

FIG. 1.—COLUMN NO. 5 AFTER TESTING. VIEW FROM WEST END.

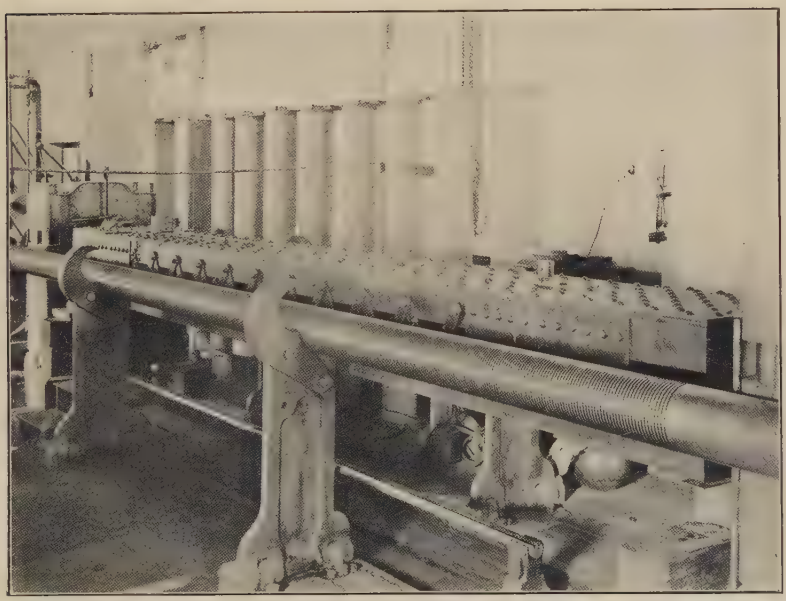


FIG. 2.—APPEARANCE OF COLUMN NO. 1 IN TESTING MACHINE.

where no part of the metal is strained beyond its elastic limit. The loads on Columns Nos. 2, 3, and 5 were maintained for different periods of time, during which intervals the amount of compression increased without advance of load. The flattened tops of the plotted curves show where loads were held in this manner, the intervals of time being 1 hour 4 min., 2 hours 2 min., and 9 min., respectively.

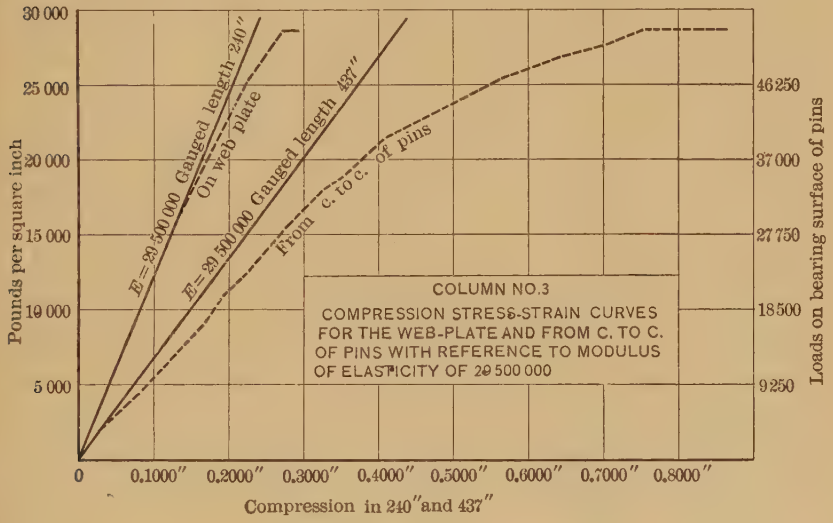


FIG. 7.

The apparent ultimate resistance of a column would doubtless be lowered by prolonged loading. In the present tests, however, the critical zone in which this result would be experienced is believed to have been but a narrow one.

Fig. 7 shows the relative compression of Column No. 3, when measured on the net section of the body and when measured from center to center of the pins.

DISCUSSION

Mr. Lucius.

ALBERT LUCIUS, M. AM. SOC. C. E.—The general facts shown by Mr. Howard's valuable paper are that the columns tested in a satisfactory manner up to the elastic limit of the material, and then one failed suddenly. The tests of the others were not brought to a conclusion.

The details of the action, under the test loading, of the plates and angles composing the column sections, are brought out clearly by the measurements and the accompanying reasoning and explanations, but the reason for the failure, at unit stress approximately representing the elastic limit, is not brought out, and yet it is the most important result of the test that the column did fail at that limit. It is true that the practical value of compression members, as parts of a structure, ceases at the elastic limit of the material, in the same way as the practical value of tension members ceases at this limit, and that, for all purposes of "sectioning," the elastic limit of the material should govern, and not the ultimate strength; but, as far as concerns the designer who is proportioning the several members of a structure, it is probably equally true that it is to him of the utmost importance to develop the design and detail of all compression members the same as of tension members, so that as wholes they not only act consistently up to the elastic limit of the material in all their detail, but hold out against ultimate failure, as wholes, to the last possible limit. The columns tested by Mr. Howard evidently did not do this.

Bearing in mind the fact that the test columns were duplicates of actual bridge posts to be connected to the trusses through gusset-plates and riveted joints, for which purposes they were probably originally proportioned, and that their change to pin-connected posts was only for the purpose of making these tests, it still appears that, even for this purpose, the pin bearing was insufficient, for it would naturally much exceed the elastic limit of the material, locally, at the pin bearings and in the plates and details of their attachments, by the time the elastic limit was reached for the sections of the post; and less than this limit can hardly have been contemplated in the tests. The proportioning of the pin bearings and attachments corresponds to practice, as represented by most of the specifications, but it is one of the points on which most of the specifications seem to be inconsistent and invite locally bad details and overstraining.

The feature which most distinguishes the make-up of the columns from the usual design is their cellular construction, the two sides, each consisting of plates and angles forming a channel shape, being stiffened and connected by transverse frames at short intervals, which made them competent, evidently, to permit the development of strains

up to the elastic limit in the section of the columns, without material deformation, and without calling in any measurable degree on the lacing, which was unusually light. To this light lacing—or in its stead, the absence of a continuous longitudinal diaphragm—the speaker attributes the cause of the early failure of the columns by crippling at or near the elastic limit. As long as the strains were within this limit, and the material maintained its full resilience, the transverse diaphragms were sufficient and the cellular posts acted properly and normally. What there was of lacing was not called upon for action; it was in a neutral condition and practically free of strain, as the tests showed; but, when the elastic limit was reached and the material became more unstable, it was ready for deformation and needed stiffening to hold the two sides of the column in shape. Then the lacing proved insufficient, and the column collapsed sooner than it should have done. As a bridge post, it was not proportioned to bring out its highest possible resistance to overload, even though it acted consistently and well within the elastic limit. Mr. Lucius.

The speaker also questions the propriety of the assumption of fact that steel in compression has the same elastic limit and yield point as steel in tension. It is probably proved sufficiently true that the modulus of elasticity is the same for both plus and minus straining, but the speaker believes the elastic limit and yield point for steel in compression to be less by an average of 20% than for steel in tension. This has been his experience in making compression tests, which, however, were carried on without the use of fine instruments, and for general information only. The Bureau of Standards and Mr. Howard can readily settle this point definitely for the benefit of the Profession.

While the speaker fully appreciates the value of the information contained in this paper, he would consider this value to lie more in the elucidation of the behavior of material under compression, generally, than in the development of facts bearing on the proper proportioning of columns. In connection with the general proportioning of columns and compression posts, the speaker believes: that the more a column deviates from the general form and properties of a closed circle or square, the less ultimate resistance will it possess; and that the radius of gyration has no meaning if there is not sufficient shear connection between the masses which determine its value. It is this amount of necessary shear connection, when the resiliency of the material becomes unstable, which column tests are hoped to furnish ultimately.

Leon S. Moisseiff, M. Am. Soc. C. E.—This paper treats of the most important problem of the day in statical engineering. The strength and behavior of large steel columns under load is a matter which vitally affects the safety of structures and the economy of their design. The results of tests on five large columns, each having cross-sectional areas of about 90 sq. in., made and reported on by a testing Mr. Moisseiff.

Mr. Moisseiff.

engineer of Mr. Howard's ability in research work, is an unusual occurrence, even in progressive American engineering practice.

It is true that the number of members tested was small, and not sufficient to enable engineers to derive rules for practical guidance in the wide field of column application, yet the minute attention paid to the behavior of the columns when tested, and the extensometer observations taken on many individual portions of the member, make these column tests especially valuable; the tests were not quantitative, but qualitative.

The paper is full of dormant suggestions, and contains the elements for a few possible studies. It is to be regretted, therefore, that the stress-strain diagrams of the material of the plates and shapes used in building up the columns (Plates XXXII to XXXVI) do not impart the information they are apparently intended to convey. The lines purporting to represent the stress-strain diagrams of the tested specimens do not seem to be drawn to any scale. The speaker has tried to establish the scale to which these curves were plotted, but has failed completely. What purpose do these forty or more curves serve if they do not convey to the reader a graphical picture of the relation of strain to stress? The value of the information presented in the paper is sufficient to warrant drawing the stress-strain curves of the component material of the columns to correct scale.

It would also be interesting to know what is meant in these diagrams by the "corrected zero line."

The compressive strain, in stepping down from pin-plate to web, was found by the author to be from 0.001 to 0.002 in. greater than that observed on the web. It developed a perfectly elastic behavior, and disappeared with the removal of the load. This may be rationally explained by the following considerations: The pin-plates, when performing their function and transmitting pressure from the pin to the web of the column, do so by means of the rivets attaching them to the web. The rivets, in transmitting the reaction to the web, are subjected to shearing strains and to bending deflections. The pin-plate itself, in distributing the pressure from the pin to the rivets, is subjected to flexure, and will develop its greatest deflection on the middle line, where the strain was observed. These three strains will combine to cause a greater displacement of the edge of the pin-plate at the middle line than that due to the compressive stresses alone.

In attempting to evaluate numerically the pin-plate strains, the following figures are obtained: The total load on the column was 800 000 lb., or one-half on each jaw. The portion transmitted—on the assumption of a uniform distribution—by the outside pin-plate to the web was 82 000 lb. There being 39 rivets provided for this purpose, each rivet carried 2 100 or 3 040 lb. per sq. in. Taking the coefficient of elasticity of the rivets in shear at 12 000 000, the displace-

ment of the pin-plate, relative to the web, due to the shearing distortion of the rivets, becomes 0.00025 in. The displacement due to the flexure of the rivets was only 0.00005. The greatest deflection of the pin-plate, however, due to its action as a distributing beam, becomes 0.00118 for a coefficient of elasticity of 29 000 000 and a depth of 22.75 in. measured on the middle line. The total strain of the pin-plate other than that due to direct compression becomes, therefore: Mr. Moisseiff.

Due to shearing strain of rivets.....	0.00025 in.
Due to flexure of rivets.....	0.00005 "
Due to flexure of pin-plate.....	0.00118 "
	0.00148 in.

or, say, 0.0015 in. This is the average of the excess in the observed strains.

Evidently, the computed strain is generally sufficient to explain the increase in strain observed in the tests. It is probable, from the indications of the observations made, that, with the metal surrounding the pin reaching higher stresses, the flexure of the pin-plate was considerably enhanced, and that to it was due the observed strain. The effect of the rivet distortion may be neglected as secondary in importance. These figures are used merely as an illustration and only to show relative values.

As the deflection of the plate is in inverse proportion to its moment of inertia, and as the latter varies with the cube of its length, the importance of a long pin-plate becomes apparent. A mere increase of length of pin-plate from its present 27.25 in. to 36 in. would result in an increase of its moment of inertia at the middle line of 2.5 times, and a corresponding decrease in its deflection. The latter would then be reduced to 0.0005 in.

Without going into refined computations, the pin-plates in the columns appear to the eye of an experienced detailer to be of insufficient length; nothing less than 36 in. should have been used.

Passing from details, and considering the results of the author's tests at Steubenville, together with those of other tests made in recent years, one arrives at the inevitable conclusion that the ultimate resistance of a well-built steel column is near the elastic limit of its component materials. No pure compression member can reasonably be expected to develop a greater resistance.

By the stress-strain curves of the column, shown in Fig. 6, the author presents bluntly the gradual development of sets characteristic of built compression members which have been exposed to the various methods of manufacturing and shop practice. The question logically arises: Where is the practical elastic limit of such columns? There being no definite point where the laws of proportionality cease, a

Mr. Moisseiff.

practical limit, based on past experiences, should be adopted, which would allow, within the "elastic limit" so defined, a certain percentage of the total shortening of the column as a set. Any excess of set would then be considered to be beyond the "elastic limit." The main value of such a conventional ruling would be to furnish a basis of comparison with tension members.

In these days of discussions on lacing bars, it is interesting and instructive to find that the strains observed by the author on the lacing of the tested columns were insignificant. A well-centered, perfectly straight column, subjected to an axial load, needs but little lacing, as long as it remains straight. Inequality in the elastic behavior of the column material is insufficient to require heavy lacing. Assume the extreme condition that the coefficient of elasticity of one channel is 24 000 000, and that of the other is 36 000 000, which values are within those observed by the author on his specimen tests: At a stress of 20 000 lb. per sq. in. of section of the column tested, the tie-plates alone would be able to equalize the unequal resistance of the channels with a shearing stress of 3 500 lb. per sq. in., and, together with the lacing bars, with a stress of about 1 500 lb. per sq. in.

The straightness of a column, however, is a relative term, and here, again, an inquiry is needed as to what would be passed on good inspection as a "straight" column, and how much deviation from a straight line would have to be allowed to meet the contingencies of every-day practice.

The results thus obtained would furnish empiric data, on which a practical column formula could be established, which would allow for possible variations in the coefficient of elasticity, reasonable lack of straightness, shop inaccuracies, and unavoidable deformation in transportation, and, at the same time, would furnish a rational basis for design of column lacing.

Meantime, before such inquiries have been made, engineers will have to use their judgment as to the proper stresses to be allowed on columns, and their reduction for increasing lengths. It may be stated here that, during the last two years, the Department of Bridges, of the City of New York, has been using in its work, for columns in axial compression, three-quarters of the stress allowed on tension members and properly reduced. Thus, where 16 000 lb. per sq. in. was allowed in direct tension, $12\,000 - 30\,\frac{l}{r}$ was used on members in compression.

Mr. Carpenter.

A. W. Carpenter, M. Am. Soc. C. E.—These tests and the record thereof, in common with some others recently published, are far in advance, in their scope and completeness, of those on which the knowledge of column strength has heretofore been based. They are not merely a determination of the load required to produce failure in the weakest part, but are a searching study of the strain action of the

Mr. Carpenter.

columns in their various parts and the metal therein, under moderate stresses and under increasing stresses up to failure, and all the results are expressed in definite measurements. They show a wonderful advance in the science of testing, and the Profession is to be congratulated in that Mr. Howard has given his attention to the column problem.

The speaker proposes to discuss this paper mainly from the viewpoint of bridge design.

Up to a very recent date, the design of these columns would undoubtedly have been considered beyond criticism. Under some very modern but widely used specifications, however, some of the details, such as the pin-plates, in respect to their length, and the lacing, would be condemned.

The specifications used in the speaker's practice require at least one full-width pin-plate on each segment to extend to the far edge of the stay-plate, and the others to some distance beyond the near edge. The pin-plates in the columns described extended about 12 in. beyond the near edges of the stay-plates, which were 2 ft. 10½ in. long. The speaker's specifications would not call for such long stay-plates, but would require one set of pin-plates to be 12 in. longer. The shortness of the pin-plates may be more clearly realized, perhaps, when it is understood that they had a length of only seven-tenths of the width of the column segments. Except in the matter of length, they fulfill the requirements of the specifications mentioned. The same specifications require the lacing to be proportioned for a shear, transverse to the axis of the column, equal to $320\ A\ \frac{r}{V}$, in pounds, in which

A = the area of the column section, in square inches;
r = the radius of gyration of the column section about an axis perpendicular to the plane of the lacing, in inches; and
V = the distance from the axis of the column to the extreme fiber, in inches.

To resist a shear of 26 000 lb., this formula would require 3 by ½-in. lacing, with 2-rivet connections. The columns in question had 2½ by ⅜-in. lacing, with single-rivet connections.

The weakness of the pin-plates was fully demonstrated by the tests, and it would appear that lengthening the plates would have strengthened the columns. There was no apparent weakness in the lacing. It must be confessed that it is difficult to see how heavier lacing could have reinforced Column No. 5 at the point where failure commenced, that is, how the buckling of the web could have been retarded by heavier lacing.

The low values obtained for the strength of these columns is somewhat surprising, in spite of the deficiencies in the details noted. The

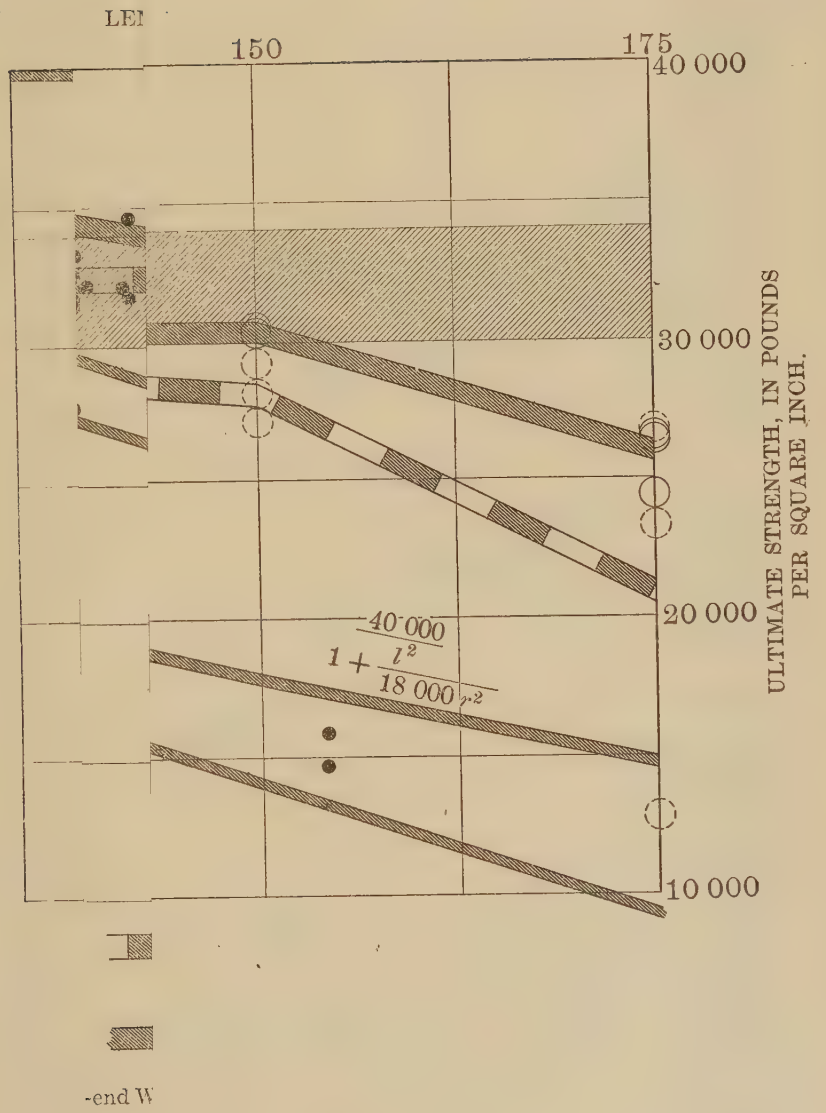
Mr. Carpenter.

speaker would have expected values from 10 to 15% higher. The values are no greater than those which have been obtained from the average of recorded tests on wrought-iron columns of structural dimensions. A closer examination seems to show that the explanation of this is in Mr. Howard's theory that the elastic limit is the element which fixes column strength, and that a column will not be much stronger than the elastic limit or the yield point of the weakest component piece. The minimum elastic limit and yield points recorded for the material of these columns are lower than the ordinary similar limits for wrought iron of the same dimensions. Viewed in this light, there seems to be no reason to expect any greater strength from columns of this grade of steel than from similar columns of wrought iron, although the ultimate strength of the steel in tension may be much greater than that of the wrought iron.

An excellent comparison of the strength of soft steel and wrought-iron columns seems to be furnished by Mr. Howard's tests at the Watertown Arsenal. His tests on wrought-iron columns were made in the period from 1881 to 1884, and on steel columns in 1908. These tests, being made on the same machine, under the direction of the same man, and on sections of comparable size although of different form, would seem to offer an unusual comparison of the two materials. The results obtained as the ultimate strengths of the columns of these series are shown on the diagram, Plate XLIII. Only the results of the tests of the pin-ended wrought-iron columns are shown; the number of corresponding flat-ended columns was comparatively small, and did not exhibit any marked difference in strength from the pin-ended ones. The wrought-iron columns were of the following sections: two channels with latticed sides; two channels with plate on one side and lattice on the other; two channels with plates on both sides ("box" section); two channels with center web diaphragms; areas 4.5 to 21 sq. in. These columns were built according to the bridge shop practice of the period, and were not particularly well detailed.

The steel columns were uniformly of four-angle and plate, I-section, as shown on the diagram; area, 13.7 sq. in.; material, open-hearth steel of 55 000 to 65 000 lb. ultimate strength. The zone of elastic limits of the steel in the steel columns, as determined from tensile specimen tests, is indicated by the cross-hatched area, this, of course, having nothing to do with the $\frac{l}{r}$ ratios. This, together with the values for ultimate strengths, and the curves representing the average values from the steel column tests, was taken from data furnished by Mr. Howard and later presented by him in a paper before the American Society for Testing Materials. The curves marked $\frac{40\,000}{1 + \frac{l^2}{18\,000\,r^2}}$.

PLATE XLIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIII, No. 1202.
CARPENTER ON
TESTS OF LARGE STEEL COLUMNS.



and $\frac{42}{16}$ $(16\,000 - 80\frac{l}{r})$ were plotted by the speaker to represent ultimate-strength curves for columns, by the well-known Gordon-Rankine and straight-line formulas, respectively. These formulas have been arranged on the basis of 40 000 lb. per sq. in. ultimate strength, for a column of $\frac{l}{r} = 0$ by the Gordon-Rankine formula, and 42 000 lb. per sq. in. by the straight-line formula. It will be noted that for values of $\frac{l}{r}$ from 40 to 80 there is little difference between the ultimate strength shown for the two kinds of material. Beyond the ratio of about 80 $\frac{l}{r}$, the strength of the wrought-iron columns is shown to fall away, generally, from that of the steel columns, although the low strength of one of the steel columns of 150 $\frac{l}{r}$ seems to indicate that the lines of strength might draw together again at higher ratios of $\frac{l}{r}$. Mr. Carpenter.

One thing on the diagram which is especially noticeable is the close agreement between the strength of the wrought-iron columns and the formula curves, and the divergence of the strength of the steel columns from these curves. This would seem to indicate that the formula curves were more correctly adapted to wrought-iron columns, and that, later, the incorrect deduction was made that steel columns would follow the same curves.

The ultimate strength of Mr. Howard's Column No. 5 is shown on the diagram by a star for the purpose of comparison.

To the speaker Mr. Howard's tests indicate that the usual working stress for bearing on pins is taken too high and should be nearer the base constant of the column formula used.

The speaker believes that the influence of form of metal is considerable in determining the strength of a column. The columns in question had nominally straight webs, more or less injured, necessarily, by straightening, punching, riveting, etc. This form is most easily deflected from a straight line by axial compression or transverse pressure. Take the same metal and corrugate it longitudinally, curve it as in the form of the Phœnix column, or bend it frequently as in Z-bars, and greater resistance to compression might be expected; in other words, it could not yield so readily.

The point of greatest interest and value in these tests undoubtedly is their bearing on the safe working stresses for columns. Assuming that we accept the lowest value for yield point in specimen tests of the material of a column as the value for the ultimate strength of a short column as a whole, what is a safe working value? The curves in Fig. 6

Mr. Carpenter.

show that the stress-strain proportionality begins to vary from about 15 000 lb. per sq. in. upward. What is the significance of this? Does it mean that repeated stresses above—even slightly above—those representing the point of divergence of the stress-strain curve from the straight line (that is, about 15 000 lb. per sq. in.) and below those causing immediate failure, would have caused ultimate failure? Would a constant stress of any value above that amount and below the stress of immediate failure have done the same? Or, does it mean that the same curve would be repeated indefinitely for loads applied and released and producing stresses nearly as high as those which were found to cause failure? The curves seem to be alike in the characteristic of a gradual and even divergence upward from the straight stem, followed by a change in direction at a point representing a value of about 25 000 lb. per sq. in. for Columns Nos. 2 and 3, and 28 000 lb. for Column No. 5. Is it not probable that if the load producing the stress of, say, 25 000 lb. per sq. in., had been removed and replaced, there would have been a slight permanent set, and the second stress-strain line would have been straight and would not be changed by an unlimited number of repetitions of this load? Until the meaning of the stress-strain curve is satisfactorily explained, it seems to the speaker that proper working values for design cannot be derived from test values.

Assuming that it is desirable to place the safe working stress for calculated stresses not higher than one-half the maximum average value which could be repeated or maintained without failure or undue distortion, and assuming that this value was 25 000 lb. per sq. in. for the columns tested, a working value of 12 500 lb. would be derived. The common straight-line formula, $16\,000 - 70\,\frac{l}{r}$ would give a value of 12 700 lb. per sq. in. for these columns.

The speaker would again call attention to the very soft grade of steel in some of the column plates. Steel with a higher yield point will give a higher column strength, as shown by the tests of Messrs. Dagron and Waddell, as presented in the Report* of the Special Committee on Steel Columns and Struts, of this Society, and the test of the Quebec Bridge model column with revised latticing. The latter test was on a medium steel column, with a ratio of $\frac{l}{r} = 25$. The ultimate strength was 37 000 lb. per sq. in. on the Phœnix testing machine, the accuracy of which has been proven by Mr. Howard by a careful calibration. The stress-strain curve drawn by the speaker from the published test results for the Quebec Bridge model column, shows a practically straight line up to 30 000 lb. per sq. in.

In order to equalize more nearly the strength of the tension and compression members in structures, it appears to be necessary to vary

* *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 401.

the basic working stress or the material, or to combine the two methods. In the matter of material it would seem desirable to grade the harder melts into the compression members and the softer ones into the tension members and, at any rate, to avoid getting material softer than the normal into the compression members. The variation of the basic stress already appears in many specifications. From the results of these column tests and others recently published, and especially on the stress-strain lines for the same, it has been urged in some quarters that the basic compression stress be taken as low as 60% of the tensile stress. This seems to be unwarranted. The average carbon-steel eye-bar, which is the only form of large tension member of which there is a sufficient test knowledge, shows an average yield point of not much more than 30 000 lb. per sq. in., and it is necessary to use the grade known as medium steel to get that. The speaker would balance the additional safety from the strength beyond the yield point in tension members against the chance of a square break and the attendant disaster which could not occur in a compression member. Stress-strain records for tension members similar to those published for columns should be brought out and analyzed. The speaker has one for a carbon-steel eye-bar which shows a divergence from the straight line very similar to that of Column No. 5 of these tests, and at about the same points. Mr. Carpenter.

Engineers should not rush to the opposite extreme, in this matter of comparative working values; they need more tests like Mr. Howard's, of both compression and tension members, and more light on the significance of the stress-strain diagram.

J. S. Branne, M. Am. Soc. C. E.—This paper is interesting to the engineer who is trying to get at the inner workings of a large built-up column subjected to steadily rising stresses; and as the phenomena observed appear to have been recorded most accurately, and in such a way that one result will help to check another, many valuable lessons may be learned. Mr. Branne.

The steel used is quite mild, having never more than 0.28 of 1% of carbon, and the percentage of impurities is low. The ultimate strength appears to rise, generally, with the increase of carbon; but Melt 23 982 forms an exception, as in this the average of fourteen tests shows a lower value than for any other melt; its elastic limit, however, is high. It would be interesting if the author would give the probable explanation for this lack of uniformity in behavior.

The action of the pin-plates under a moderate compression, not exceeding 9 000 lb. per sq. in. in the body of the column, is interesting as well as characteristic. The pin-plates, together with the column webs, take the applied load. At or near the pin it is evident that the unit stress, while not uniform over a plane normal to the axis of the column, as plainly indicated by the strains, is locally uniform in the pin-plates and column body, all receiving equally the stress from the

Mr. Branne. pin, on the surface of the column, almost a smooth body. Receding from the pin, the unit stress in the column body will rise, due to the transference of the pin-plate stress through the rivets and by friction. At or near the last rivets in the pin-plate, this difference in unit stress will have its maximum value; hence the web from the pin to the end of the pin-plates is strained more than the pin-plates, and a small difference is detected, indicating an elastic motion of the pin-plates relative to the web.

Fig. 2, Plate XXXVII, indicates a unit stress of 5 700 lb., and while the web cannot be stated by measurement in the same 10 in., yet in 30 in. it averaged 9 600 lb. When the test load was removed, this motion was reversed, and the set disappeared. It may be that when there are more and thinner pin-plates, this motion will be smaller, or the ideal condition would be approached of having just enough metal in the pin-plates at all points to make the unit stress the same as in the body of the column.

The local failure of Columns Nos. 3 and 4 might have been prevented by a long diaphragm, with its web parallel to the axis of the column and reaching close to the pin, thus minimizing the small bending moments in the two component parts of the column, preceding the dishing, as shown on Plates XL and XLI.

Referring to the action of the lace-bars, the tests show that when the force is applied symmetrically with respect to the axis, or rather a force coinciding with the same, as in a testing machine, and with short and stocky columns, small lace-bars, simply balancing the component parts, so to speak, and preventing any first set, are quite ample, even for a column having two component parts of such magnitude as 45 sq. in. each.

Mr. Rights. Lewis D. Rights, M. Am. Soc. C. E.—This Society is fortunate in securing a paper from a man so well versed in the science of testing metals as Mr. Howard. An examination of the records of column tests show that he was connected with all the important tests at Watertown Arsenal, and that he had charge of the tests of the eight large columns in connection with the calibration of the Phœnix machine. In a recent paper, "The Pittsburg and Lake Erie Railroad Cantilever Bridge Over the Ohio River at Beaver, Pa.,"* mention was made of the determination of stresses by gauge measurements. These measurements are being made, under Mr. Howard's direction, with gauges of the same type as those used on the columns described in his paper.

The reader's attention is attracted at once by the size of the columns. They are the largest ever tested to failure, having a cross-sectional area of 95 sq. in. Mr. Bland is to be commended for the broad spirit he has shown in spending a considerable sum of money to

* *Transactions*, Am. Soc. C. E., Vol. LXXIII. p. 136.

determine the character of the structures under his direction. He is also to be congratulated on being able to secure the services of Messrs. Buchanan and Howard, who conducted these tests, as their experience guarantees observations calculated to show a large number of interesting features. It is to be hoped that other large users of steel will follow Mr. Bland's example, and that much may be added to the data concerning steel columns. Mr. Rights.

Passing by many interesting features concerning the action of the details under stress, one important thing is the matter of local strains. The author mentions three conditions which affect the results, namely, cooling strains, initial strains due to straightening, and strains due to punching and shearing. Engineers have long recognized these conditions, and, in connection with columns, have endeavored to assign some arbitrary value which will take account of them. J. M. Moncrieff, M. Am. Soc. C. E., in a paper on "The Practical Column Under Central or Eccentric Loads,"* dwells quite fully on the subject.

According to the speaker's experience, about 95% of all the channels and 60% of all the angles which come from the average rolling mill have to be straightened after they are cold. Most of the work is done in a gag press, and it is evident that, in order to straighten a member, a small portion of it must be strained beyond the elastic limit on both sides. The side which was concave has been stretched in tension, and the side which was convex has been compressed, so that these two edges have been worked beyond the elastic limit, and have a considerable amount of initial strain. Now, when this material is placed in a column, it is also evident that the compressed side has comparatively little resistance to additional loads. This condition, of course, probably only applies to a thin section of the outside metal; nevertheless, it weakens the column by that amount.

The initial strains produced by punching the metal are not always recognized by engineers. It is well known by shop men that a long angle punched in both flanges will show a considerable lengthening, due to the punching, and, if this is not symmetrical, the piece will generally have to be straightened after being punched. The speaker has often had occasion to observe long channels, say, from 30 to 40 ft., which were intended for columns, and were punched throughout their length in both flanges. The punching tended to elongate the channel along the rivet gauge line. Naturally, the web, which contained the bulk of the material, was not elongated, and the result was that the channels were bowed to the extent of 2 or 3 ft. These channels had to be straightened before they could be assembled, and it is evident that there was considerable internal disturbance of the metal. Another feature of punching is the local disturbance in the vicinity of the punched hole. Some specifications call for the removal of $\frac{1}{16}$ in.

* *Transactions*, Am. Soc. C. E., Vol. XLV, p. 334.

Mr. Rights. of the metal around the hole by reaming. This, no doubt, does remove the badly injured material, and smoothes out slight differences between the punch and the die. If, however, one takes the trouble to rust the metal around the punched hole, the stress lines can be seen running in spirals away from it. It is probable that these stress lines do not extend into the metal very deeply, but they indicate that engineers are not dealing with an ideal material, and that the strength of columns is based on a number of such conditions, which are not always fully recognized.

Fig. 6 shows the stress-strain curves for three of the columns. It will be noted that the author has recorded a seeming departure from the line of proportionality at about 20 000 lb. per sq. in. He seems to think that this is due to the various local strains which he has mentioned, and which probably occur in all columns. The speaker agrees with Mr. Howard, and believes that if extremely delicate instruments could be secured, these local adjustments would show at even lower values than 20 000 lb. He does not consider, however, that this may be interpreted to mean a complete departure from the elastic conditions at these low stresses. Probably, as the minor local conditions in the component parts are adjusted, those portions which were in tension take on additional loads and relieve the compressed parts. The speaker believes that if a new set of gauge measurements were made on the column after it had been stressed up to 25 000 lb. per sq. in., the results of the local failures would no longer be apparent, and the column would show a strict proportionality up to about 27 000 or 28 000 lb., which, as indicated on Fig. 6, seems to be the yield point.

Mr. Worcester. J. R. Worcester, M. Am. Soc. C. E.—The speaker desires to express his sense of obligation to the author and to Mr. Bland for the information contained in this paper.

The paper is important for several notable reasons. It throws light upon the relation of large-sized tests to small tests. In the past many tests have been made on small specimens, but the number on large specimens is so limited that every additional one helps.

Another point which, of course, is very prominent in the paper, is the extreme refinement of observation which characterizes it. It is doubtful whether as many measurements of deformations have ever been made as were made in this particular instance.

To be sure, many of the results bear out what might have been expected. On the other hand, there are some which are quite surprising, and rather difficult to explain; but they all tend to show one important fact which is apt to be overlooked, namely, that there are parts of a built-up member which are stressed far beyond the average stress. Engineers are apt to think of the stress applied to the area of the column section as being uniform throughout. Considering the results of these experiments, however, they must bear in mind, that

the distribution of stress is not uniform, but that if the total stress is divided by the area, the average result by no means represents the maximum. Mr. Worces

Another important point is shown by the stress-strain diagrams, which clearly indicate the low stress at which the proportionality between stress and strain seems to be lost. The exact explanation of this may be clearer after more tests have been made, though Mr. Howard's analysis undoubtedly helps to explain it.

It is sincerely to be hoped that the author's example will be followed, and that others who have the opportunity to provide full-sized tests will see their advantage to the Profession.

HORACE E. HORTON, M. AM. SOC. C. E. (by letter).—In discussing this paper the writer wishes to express, to Mr. Howard and his colleagues of the Bridge Department of the Pennsylvania Lines West of Pittsburg, his obligation for their contribution of the very large proportion of all the research work, of record, bearing on this subject. Mr. Horton.

On another occasion, in discussing the compressive member, the writer said:

> "It is subtle and illusive, and may side-step."
> "It is composite, and not homogeneous."
> "No law is written as to the proper relation and proportion of its constituent parts."

Here he will add: There is no clear and definite knowledge of how to get the greatest efficiency out of the material used in a wrought-metal, composite, compressive member.

Lest the writer should repeat himself, it seems best to refer to his discussion of the paper by J. R. Worcester, M. Am. Soc. C. E., on "Safe Stresses in Steel Columns."*

Mr. Howard's tests show ultimate strength in compression of only one-half the ultimate strength in tension of the same class of material. If it is assumed that the elastic limit in tension is one-half of the ultimate strength, and that the rational working load is one-half of the elastic limit, then the ultimate working load is one-quarter of the ultimate tension. If the ultimate strength in compression is one-half of the ultimate strength in tension, and the elastic limit in compression is three-quarters of the ultimate strength in compression, then the ultimate working load in compression would be three-eighths of the ultimate strength in tension; that is, the working load in compression is three-quarters of the working load in tension.

In connection with Mr. Howard's paper, it seems well to call attention to the Progress Report of the Special Committee on Steel Columns and Struts,† showing results of tests on 263 full-sized members. These, with Mr. Howard's five tests added, make 268 in all:

* *Transactions*, Am. Soc. C. E., Vol. XLI, pp. 178 to 186.
† *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 401.

Mr. Horton.

184 on riveted iron members;

52 on steel homogeneous sections 25 $\frac{l}{r}$ or more (pipe and **H**-figures); and

32 on riveted steel members.

The ratio of the ultimate compressive strength (also the elastic limit on pipe, **H**, and riveted-steel members), shown by these tests, to the ultimate tensile strength of the material, is indicated in the diagram, Fig. 8.

The 184 tests on riveted iron members are in a sense obsolete, and with the 52 pipe and **H**-sections—236 tests in all—are useful as an index of the decreasing unit value in compression as $\frac{l}{r}$ increases. The 52 homogeneous tests of pipe and **H**-sections are further useful in showing the probable full efficiency of the steel compressive member. It is interesting to note that the pipe and **H**-sections, in plating, followed essentially the same line.

Of riveted-steel compressive members, built up of parts riveted together—composite, not homogeneous—there are only 32 tests, from 25 to 97 $\frac{l}{r}$.

The average ultimate strength of compressive members, as shown by Fig. 8, 25 to 97 $\frac{l}{r}$, for pipe and **H**-sections, is from 75 to 50% of the ultimate tensile strength; and it is from 60 to 43% for riveted sections.

The average elastic limit of compressive members, as shown by Fig. 8, relative to tensile strength, for pipe and **H**-sections, is from 52 to 47%; and it is from 45 to 20% for riveted sections.

This is both interesting and instructive, because it shows, for riveted members, only 65% of the elastic limit of the homogeneous ones, that is, 65% efficiency—35% loss in the composite riveted member. Considering that engineers, from the nature of their work, are obliged to use riveted members, surely it is desirable to raise the efficiency above 65 per cent. How is it to be accomplished?

Referring particularly to the five tests by Mr. Howard, it is an interesting speculation to inquire what would have developed if:

(*a*) The members had been exactly as tested, except that twice as many $\frac{7}{8}$-in. rivets had been used;

(*b*) Same as (*a*), but with twice as many $\frac{7}{8}$-in. rivets (four times as many $\frac{7}{8}$-in. rivets as in the member tested);

(*c*) If the members had been exactly as tested, with same number of rivets, but with the rivets increased to $1\frac{1}{4}$ in. in diameter;

(*d*) Same as (*c*), but with double the number of $1\frac{1}{4}$-in. rivets;

Mr. Horton.

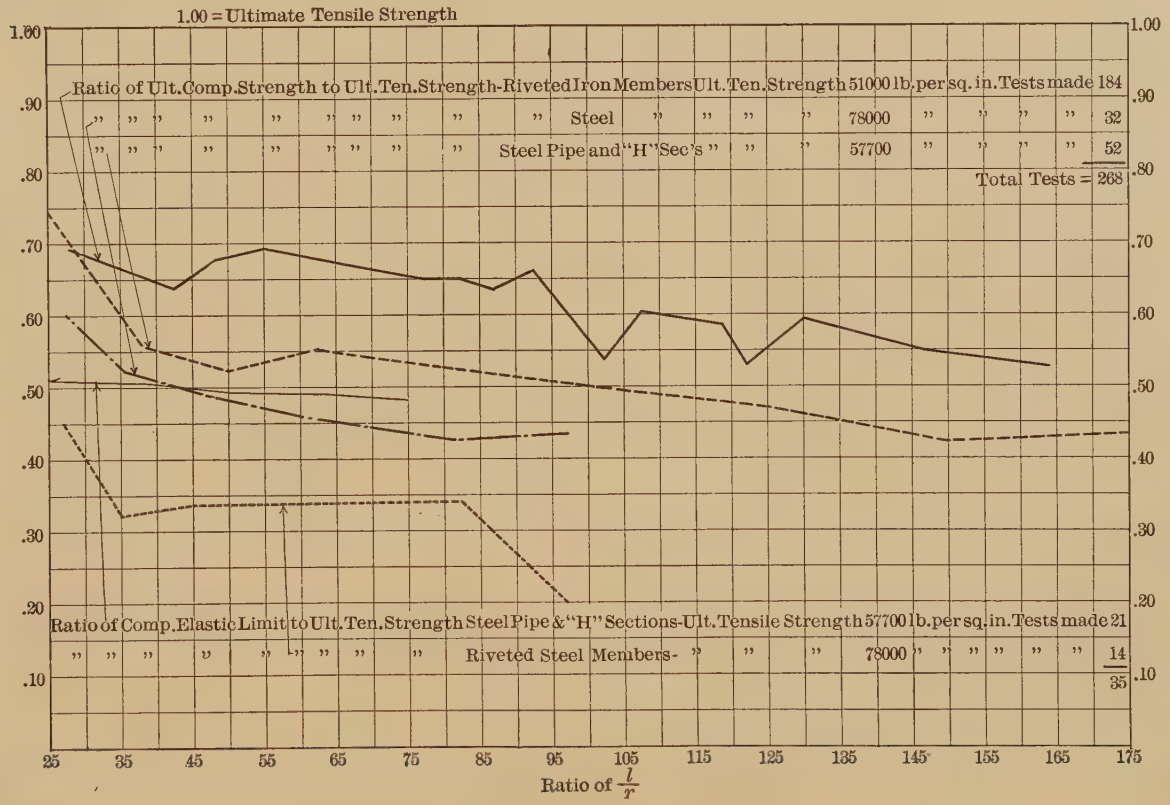

1.00 = Ultimate Tensile Strength
Ratio of Ult. Comp. Strength to Ult. Ten. Strength - Riveted Iron Members Ult. Ten. Strength 51000 lb. per sq. in. Tests made 184
" " " " " " " " " " " Steel " " " " 78000 " " " " 32
" " " " " " " " " " Steel Pipe and "H" Sec's " " " 57700 " " " " 52
Total Tests = 268
Ratio of Comp. Elastic Limit to Ult. Ten. Strength Steel Pipe & "H" Sections - Ult. Tensile Strength 57700 lb. per sq. in. Tests made 21
" " " " " " " " " " Riveted Steel Members - " " " 78000 " " " " " " 14
35
1.00
.90
.80
.70
.60
.50
.40
.30
.20
.10
25 35 45 55 65 75 85 95 105 115 125 135 145 155 165 175
Ratio of $\frac{l}{r}$

FIG. 8.

Mr. Horton.

(*e*) If the compressive members had been built of four 6 by 6 by 1-in. angles and two 28 by 1-in. plates, without diaphragms, $\frac{7}{8}$-in. rivets, the same tie-plates, the same lattice, in fact, the columns of the same weight as those tested;

(*f*) The same as (*e*), but with twice as many $\frac{7}{8}$-in. rivets;

(*g*) The same as (*f*), but with double the number of $\frac{7}{8}$-in. rivets (four times as many $\frac{7}{8}$-in. rivets as in the member tested);

(*h*) Same as (*e*), with $1\frac{1}{4}$-in. rivets, the same number as in the member tested;

(*i*) Like (*h*), but with double the number of $1\frac{1}{4}$-in. rivets.

This would give nine modifications, added to the test already made, making ten; with five of each class, equaling 50; and 20 changes of $\frac{l}{r}$, extending from 25 to 200, would require 1 000 test pieces and tests.

If the length, breadth, thickness, and all component parts of these pieces were of one-eighth size, each unit would weigh $\frac{1}{5\,1\,2}$ of each member described by Mr. Howard, and it can be safely concluded that the cost of the material for the 1 000, ready for the testing machine, would be less than that of the five units tested.

The $\frac{1}{8}$-dimension test member representing (*e*) would be made up of four $\frac{3}{4}$ by $\frac{3}{4}$ by $\frac{1}{8}$-in. angles, two $3\frac{1}{2}$ by $\frac{1}{8}$-in. plates, and $\frac{7}{6\,4}$-in. rivets. For (*h*) $\frac{5}{3\,2}$-in. rivets would be used.

It is unfortunate that the Special Committee on Columns and Struts has gone on record as indicating a want of confidence in "Laboratory Tests." What is a laboratory test? The engineering press has called attention to a design of a structure wherein a compressive member having a cross-section of 2 037 sq. in. is contemplated. Mr. Howard's 90-sq. in. sections are essentially one-fifth of this. In some of the tests reported by the Committee the members are less, in each of the three dimensions, than one-third of the dimensions of other test members under consideration; hence a member of $\frac{1}{3}$ dimension is not a laboratory test specimen because the Committee has recognized it.

The smaller cross-sections in the Committee's report bear a relation to the proposed 2 037-in. section of quite one-sixteenth for three dimensions, that is, the Committee has recognized as rational-sized tests, members so slight, as compared with the member of 2 037-in. cross-section, that the weight of a unit of each, with the same $\frac{l}{r}$, is as 1 to 4 096. The writer has suggested a $\frac{1}{8}$ dimension of Mr. Howard's 90-in. section. This would be approximately a $\frac{1}{3}$ dimension of the smallest size the Committee has recognized.

Does the Committee believe that there is concealed about any engineer knowledge of the compressive member equal to what might be

expected to develop from 1 000 so-called laboratory tests of ⅛-dimension pieces along the lines indicated above? Mr. Horton.

From what source have engineers gathered their knowledge of the compressive member? Large numbers of laboratory tests have been made on small solids and small pipe, all homogeneous sections, which are largely the source of all present knowledge as to the compressive member. The writer's appeal is that the research be extended by including tests of large numbers of composite, riveted, wrought, compressive members, which may be handled in a 100 000-lb. testing machine.

The engineering schools have testing machines of sufficient capacity to handle the specimens herein indicated. They have ambitious professors and zealous students. With slight encouragement, on the part of the Committee, a series of tests could be carried out which would give definite knowledge, and, the writer believes, would indicate the way in which the efficiency could be raised to more than 65 per cent.

Arthur N. Talbot, M. Am. Soc. C. E. (by letter).—These tests are noteworthy in many respects, particularly in that a study was made of the action of the column in its different parts at loads below the ultimate. Tests to destruction may not of themselves have much meaning. The author is also to be commended for his interesting and valuable discussion of the data of the tests. Mr. Talbot.

The writer is not entirely satisfied with the conclusion that the elastic limit of the material in tension has a controlling influence on the ultimate strength of the column. Throughout the paper the author seems to assume that length of column does not affect the strength or modify the distribution of stress for any load. Although the trend of evidence in available tests is that, within ordinary limits of length, column action does not follow the law which is assumed in the derivation of the old column formulas, yet the consensus of experimental data indicates that for ordinary lengths of column there is a noticeable decrease in strength with an increase in length or slenderness ratio. Tests made at the University of Illinois also indicate that the effect of length on deformation is greater at the higher than at the lower loads. How much of the loss in strength is due to increased opportunity for non-homogeneity of materials, non-straightness of pieces, and non-integrity of section cannot be told. However, from the experimental data which are available, it may be expected that a column of the length of No. 5 will be weaker than one of half this length.

Elastic limit of material is an elusive thing, not easily defined, and the finer the measurement the lower the stress at which set and lack of proportionality of stress to elongation may be found. The effect of exceeding the elastic limit of steel has not yet been determined. To exceed the yield point in any part of a structural member is a more serious matter; the distribution of stress is at once modified, and a

Mr. Talbot.

larger stress is laid on other portions of the cross-section. The measurements in the tests were not numerous enough or sufficiently distributed over the length and cross-section of the column to determine how much the maximum local stresses were in excess of the average stresses; but, although the column tested was of very stocky section, it seems almost certain that the average stresses must have been greatly exceeded in places, even before the yield point of the metal was reached anywhere. The passing of the yield point locally may account in part for the deviation of the stress-strain curve from the straight line in Fig. 7. Such a variation from the straight line is to be found in the test of any built-up column. To the writer it seems that the evidence of the tests, taken in connection with phenomena observed elsewhere, indicates that the yield point of the metal must have been exceeded at different points in the column at loads below the ultimate load, and that the fact that the average stress at failure was about the same as the elastic limit of the softest metal is only a coincidence.

The author's comment, in connection with the increased intensity of stress in the vicinity of the pin bearings, that "details of construction generally exert an influence on the behavior of the member, and not infrequently modify the ultimate resistance materially," is worthy of emphasis. Too often designers think the problem is solved by the mere provision of sufficient metal, without any consideration of the manner in which the stress may be transmitted and distributed. Before a pin-plate can transmit stress to the web-plate, there must be a difference in deformation in the two plates, particularly if there is no movement of the pin-plate over the web-plate. If there is slip (and if we are to judge from the tests of the riveted joints, a movement must have occurred long before the full loads on the columns were reached), then there will also be a deformation or detrusion of the rivets to transfer stress from plate to plate, and this transfer of stress will take place at those portions of the pin-plate well away from the pin. The tests indicate that this movement of the plates and detrusion of rivets were present. They would also seem to be the cause of the difference in measurements, rather than any bending of the pin.

The writer is glad that the author recognizes the effect of time on the action of a compression piece stressed locally beyond the yield point. If a longer time were given in tests of short pieces, quite different data would be obtained.

It would have been interesting if systematic measurements had been made at various points along the length of the column, and on all sides of the component members, to ascertain the amount of local wrinkling or buckling and also to determine the stress in all lattice bars. It would also add to the completeness of this excellent paper, if the author would give a record of the permanent set found in the columns.

Mr. Christie.

JAMES CHRISTIE, M. AM. SOC. C. E. (by letter).—These columns were carefully designed, and, doubtless, carefully constructed; and the minor disturbing features, referred to by Mr. Howard as due to unequal cooling, cold-straightening, etc., probably would not reduce their resistance seriously as a whole.

There are reasons for believing that when $\frac{l}{r}$ equals about 100, the resistance to compression of a column, per unit of its section, should be approximately equal to the elastic limit of its material, and that columns having the same ratio of length to section as those tested, should develop a maximum resistance considerably exceeding that of the elastic limit of the material. As the recorded resistance of these columns fell a little below the said elastic limit, it would be interesting to know how much, if any, of this was due to insufficient area of pin bearing. Mr. Howard intimates that this may have been a source of general weakness. Under the maximum compression the bearing areas for the pins were stressed much higher than the elastic limit of the material, and probably nearly as high as its ultimate tensile strength.

This excessive burden was indicated by the failure of the reinforcing pin-plates and the local disturbance at these ends, and probably contributed toward the general failure of Column No. 5. As Mr. Howard well remarks:

"Regularity of action is limited to stresses within, and ceases at, the elastic limit, and when over-straining of the metal occurs, locally or in general, the strength of the column is menaced."

Mr. McLure.

N. R. MCLURE, ASSOC. M. AM. SOC. C. E. (by letter).—Each intelligently made test on large-sized, built-up, steel, compression members adds to the all too scant knowledge of their action; and the publication of the results of these tests, in order that interested engineers may study them and draw conclusions therefrom, cannot be too highly commended. The author has done a great service in adding to the small stock of information on this subject.

In the five tests which form the basis for this paper, the average ultimate strength of the columns, as shown in Table 1, is about 29 500 lb. per sq. in. The average elastic limit of test specimens for angles and plates composing these five columns, taking the value of the angles as 31% of the total cross-section of each column, is about 32 300 lb. per sq. in. As the elastic limit in a test specimen will usually run about 10% higher than that of the full-sized piece from which it is cut, the average value for the elastic limit of the material entering into these columns may be taken as about 30 000 lb. per sq. in., which is practically equal to the average ultimate strength of the columns. In that point, these tests agree with those on other full-sized columns which the writer has noted during the past three years, and, in fact, it has almost proved a rule without exception that the

Mr. McLure.

ultimate strength of any latticed steel column, with properly designed details, and a ratio of $\frac{l}{r}$ of less than 60, is about equal to the elastic limit of the metal composing the cross-section of that column. It is evident, from several remarks in the paper, that the author has recognized the influence of the elastic limit of the metal on the ultimate strength of the column.

The number of points covered in the tests on these columns, and the opportunity given, by the numerous measurements of local strains, to study the detailed action of the load throughout the member, make them doubly interesting, and emphasize the desirability of establishing a standard for conducting tests on full-sized compression members, so that the necessarily few tests made from time to time, as occasion requires, may be grouped and tabulated for comparison. It seems to the writer that the era of full-sized tests on steel compression members is just beginning. With the new testing machines now in course of completion, the apparatus available for making tests will stimulate interest in them, and a continuously increasing study of the action of built-up steel compression members may be expected. It seems, therefore, that the Society could further the value of such future tests in no better way than by recommending a general standard method for conducting them, including the desirable observations to be made and other details which, if adhered to, would bring all tests to a suitable basis for comparison. By so doing, doubtless in a few years the old column formulas, based on the ratio, $\frac{l}{r}$, might be replaced, or at least supplemented by more rational ones applicable to the various types of built-up steel compression members.

The author's observations in regard to local strains are most interesting, particularly the agreement of the actual with the theoretical ratio of lateral expansion to direct compression.

Each new test of this sort only emphasizes the important and modifying influence of details of construction on the final result, and this should serve as an incentive to all designers to spare no effort in working out details which, without an atom of doubt, will develop the full compressive strength of the member.

The influence of initial strains, referred to by the author, is one which cannot readily be taken account of in design, except by allowing a sufficient margin of ultimate strength over working load. Annealing may at times be resorted to, but the elastic limit of the metal is thereby reduced, which reduction might be more serious than the presence of local zones of over-strained metal. Punching, shearing, straightening shapes while cold, and other manipulation of the steel during fabrication and erection—including the use of drift-pins—are all necessary evils in the present state of the art, the effects of which will always

be uncertain. By numerous tests, similar to those recorded in this paper, however, we may hope gradually to increase our knowledge and better our results, and the writer wishes again to emphasize the importance of conducting such tests according to a definite standard, and of publishing the results for the benefit of the Engineering Profession. Mr. McLure.

R. S. Chew, Assoc. M. Am. Soc. C. E. (by letter).—This is a most valuable paper, as it presents in detail the actual strains in compression members under load, and emphasizes the sources of their weakness. Mr. Chew.

The writer believes that the column formula will eventually take account of the imperfections which really exist in built members, namely:

First.—Initial stresses in material due to manufacture,
Second.—Variation in strength of component parts of section,
Third.—Crookedness of component parts,
Fourth.—Crookedness of whole member,
Fifth.—Local stresses due to details and shop work,
Sixth.—Accidental eccentricity,
Seventh.—Deflection caused by the foregoing imperfections.

These items, with one or two exceptions, have been covered by Mr. Howard's tests.

Referring to the variation in strength of the component parts of the section, it is seen that in Table 5 the author has given the elastic limits of the shapes composing the section (it is to be regretted that he did not give the actual location of these various pieces in the section). In Column No. 5, if it is assumed that the pieces of lower elastic limit are grouped in one half-section (a condition which might exist), the resisting axis is 0.178 in. removed from the center of gravity. Column No. 5 is of excellent proportions, and the eccentricity due to this item is small; still, the idea that a perfectly straight compression member with the load applied at the center of gravity will develop a definite bending moment, independent of $\frac{l}{r}$, is important. It can be readily seen that a latticed channel column with a 25% variation in elastic limit would have a high eccentricity.

The writer is heartily in accord with the author in the statement:

"If it is accepted that the elastic limit has a controlling influence on the ultimate strength of a column (as it is believed to have), then variations of from 25 to 27% in this value, as witnessed in the tests of the metal of these plates and angles, would overshadow those considerations which find expression in empirical formulas for strength, and take no account of such features."

The suggestion of remedy in designing work, however, would be to assume the existence of a variation in the elastic limit between the

Mr. Chew. two halves of a section, locate the resisting axis, and provide for this bending moment.

It is curious that the author makes no note of the condition of the specimens with reference to crookedness. It is also strange to find that standard specifications simply require that such members shall be straight and true. It is a fact, however, that they are not straight, and the writer believes that a limit of departure from straightness should be specified. Those in charge of one of the leading testing bureaus state that commercial columns, 30 ft. long, fabricated under the best conditions, have very seldom less than $\frac{1}{4}$ in. of kink or bend in them. The fact that Column No. 5 failed at the one-third point, instead of at the middle, leads to the belief that there may have been a kink near that place.

The item, local stresses, is interesting. The author's investigation shows the importance of good details. However, the statement that there is a contractile force of 30 000 lb. per sq. in. for riveting is not in accord with the writer's experience, for he has noted cases of heavy structural members, inspected and passed at the shop by competent men, in which a number of rivets have worked loose during shipment. This would seem to indicate that the rivet in tension was strained beyond the elastic limit, so that the contractile force would be much less than the author has assumed.

The writer understands that the investigation with reference to the stresses in lattice bars was made at the middle of the column length. Although these bars may help to distribute the load, to a limited degree, and also tie the sections together, yet one of their main functions is to carry the transverse shear produced by any bending moment in the column, which shear is zero at the middle. It would be interesting to know how these stresses vary toward the ends.

The writer believes that the investigations described in this paper will be a great aid in the development of a column formula. It may or may not be agreed that the six items cited by the writer produce the mysterious bending moment in columns; however, if a limit for the variation in the elastic limit—degree of straightness—can be defined, it is believed that a formula including the moment caused by these items would approximate to working conditions.

Mr. Cole. George N. Cole, Assoc. Am. Soc. C. E. (by letter).—One of the discussors has mentioned the desirability of more delicate instruments for the measurement of changes in large steel columns under load. The writer has noted the description of Mr. Howard's application of the micrometer screw, and believes that small mirrors might be used in testing large bridge members. Some of these mirrors might be attached directly to the column and others could be set up on a separate axis and made to rotate by any motion of the column.

By this means the bending or twisting of the member could be

observed in a state of rest and under load, it being understood, of course, that a fixed source of light is furnished for the mirrors, and that graduated scales are placed at the right distances to magnify properly all motions of the beam and render them calculable. Mr. Cole.

The writer understands, of course, that this method has not the continuous possibilities of Mr. Howard's ingenious centering for his micrometer screw, but it is thought that it might have an application in certain of the studies of column tests, and for movements under load.

F. P. Shearwood, M. Am. Soc. C. E. (by letter).—Is the safety of a column to be estimated entirely by the ultimate load which it will carry under ideal conditions? When designing a compression member for any structure, engineers know, within fairly close limits, the maximum axial stress which can come upon it, and if it were only the possibility of the axial stresses exceeding those calculated, there would be no occasion for the large factor of safety called for by modern practice. The extra strength in steel structures is required for the purpose of guarding against shortcomings in the material or workmanship, maltreatment of the members, or possible secondary strains and stresses resulting from various causes. If one or more of these defective conditions is present, it is very evident that the strength of columns under ideal conditions is altered. To experiment solely with columns in apparently perfect condition is scarcely a certain or satisfactory method of determining the relative safety of different types of compression members when performing their customary work. The presence of bent lattice-bars, and even kinked flanges, in almost any steel structure, shows that it is never safe to assume that steel members can be relied on to be in perfect condition when called on to perform their duty. Mr. Shearwood.

If two columns are compared, the one with its component parts of sturdy dimensions and continuously connected, and the other with its material distributed and intermittently braced together so as to obtain a large radius of gyration, the latter will be less capable of resisting abuses and will be more sensitive to weakening influences. Should it be designed, therefore, to carry the greater unit stress because under ideal conditions it will resist the greater test load?

Charles Worthington, M. Am. Soc. C. E. (by letter).—There are two salient features of these tests: first, the elastic behavior of the columns in the 240-in. length, and second, the insufficiency of the pin bearings. The first of these is reassuring to the designer in showing, by actual test on large compression members of a usual type, that the member as a whole is perfectly elastic through a range of stress to which he would limit its use, and practically elastic for twice this limit. By the frequently used compression formula, $\left(16\,000 - 70\,\frac{l}{r}\right)$, Mr. Worthington

Mr. Worthington.

the tested columns, 2 to 5, inclusive, would be limited to a working stress of 12 700 lb. per sq. in., and a glance at Fig. 6 shows that these columns (the three for which records are given, at least), are practically elastic up to a unit stress of about 25 200 lb., or nearly twice this allowed unit stress; and that above this the stress-strain diagrams become irregular. With the usual tests now made on the material in such members, there would seem to be little danger of developing in practice any serious deviation from the results here obtained.

Comparing this with the usual practice of allowing a working stress in tension of 16 000 lb. per sq. in., or about one-half the elastic limit of the material in tension, it is evident that this practice in designing gives, as it should, an equal rating to tension and compression in reference to the respective elastic limits of the members, which, after all, are the designer's real ultimate limits.

Referring now to the second feature of importance, the defective pin bearings: There are included in these tests ten identical, though separate, pin bearings, which show great similarity in their action under load and a pronounced weakness as compared with the body of the column. Moreover, the bearing pressure on the pins is practically that permitted under the usual practice, and the working stresses in the rivets connecting the pin-plates to the webs are also in accordance with the usual practice. Assuming the working stress in compression at $16\,000 - 70\frac{l}{r}$, the working loads for Column No. 1 and Columns Nos. 2 to 5, inclusive, would be 1 285 000 and 1 143 000 lb., respectively. The pin bearing is 48.75 sq. in. in all cases, giving working pin pressures of 26 360 and 23 450 lb., respectively, while in usual practice the working pin pressure is taken at 24 000 lb. The results of these tests show that this value is very much too high to be used in conjunction with values of 16 000 lb. in tension and $16\,000 - 70\frac{l}{r}$ in compression.

Various interpretations have been placed on these tests, by the author and by other discussors, and especially on the action of the pin-plates.

The records of the tests are presented in a clear-cut and altogether satisfactory manner for the consideration of others, though in a matter of such importance the writer would like to see recorded a complete set of strain measurements for each of the columns, just as was done for Column No. 1; especially the strains in 10 in. along the center of the column web, as these are, of course, the principal strains in the member.

There is one most important characteristic of the strains measured in 10 in., which the author does not bring out, namely, the variation in the amount of strain over the space in which it is measured. He

Mr. Worthington

uses the strains directly as measured, which amounts to using the average strain in the space, whereas it is the maximum strain which is of real importance, as that governs entirely when near the elastic limit, and really determines the elastic limit and ultimate strength of the member. For example, in Fig. 2, he gives a stress of 11 210 lb. for the stress just in front of the pin, whereas this is only the average stress in the first 10-in. space in front of the pin; the actual stress at the pin end of this 10-in. space would be much greater, and at the other end less, than the 11 210 lb. shown. It will be seen that, along with the strain of 0.0038 in. in the first 10 in., noted on Fig. 2, Plate XXXVII, there is recorded a strain of 0.0019 in. in the second 10 in.; assuming for the present a uniform rate of variation in the strain throughout the two 10-in. spaces, this rate of variation would amount to 0.00019 in. in 1 in., which would give at the pin end of the first 10-in. space a strain per inch of 0.000475 in., corresponding with which would be a stress of $11\,210 \times \frac{0.000475}{0.000380} = 14\,010$ lb. at that point, as against the average stress in the 10 in. of 11 210 lb. given by the author. The author's only apparent recognition of this variation is in giving the title, "mean stress," to Fig. 2. It has a most important bearing on the whole problem, as the writer will attempt to show.

Mr. Moisseiff, by a series of calculations, apportions the major part of the recorded movement of the outer pin-plates to "flexure of pin-plate," but these calculations seem to consider the pin-plates alone, disregarding the modifying influence of the continuous webs on the flexure of the pin-plates. The total thickness of the webs is $2\frac{5}{8}$ in., as against a total thickness of only 1 in. for the outer pin-plates, and the depth of the webs as a beam is the whole distance between the pins at each end of the column, as against a depth of 22.75 in. used by him as the depth of the pin-plates in calculating the amount of their flexure. This enormously deep beam, to which the outer pin-plates are intimately coupled by rivets and by the pin itself, would certainly govern the amount of movement, if it were a matter of flexure. Moreover, if flexure played as large a part as indicated by Mr. Moisseiff, it would be expected to show in an increase of the transverse strains at the end of the pin-plates farthest from the pin, whereas in neither of the cases recorded does this occur; in fact, there seems to be a tendency to contract at this point rather than to expand. To the writer the records of the test seem to indicate that flexure plays a very small, if not an altogether negligible, part in the results.

If the pin-hole had been bored through the pin-plates and webs simultaneously after the column had been riveted up, and if the pin was perfectly straight—both reasonable assumptions in this case—the pin would bear against the pin-plates and webs simultaneously, and

Mr. Worthington.

any forward movement of the point of contact between pin and pin-plate would be accompanied by an equal movement of the point of contact between pin and web. A different assumption involves as a fact, either the bending of this short 10-in. pin, or the formation of an offset or shoulder on the pin; neither of these conditions is reported in the record of the test, nor do they seem at all probable or even possible.

Based on this assumption, then, the recorded strains along the center line of the column may be interpreted as follows: The first and second are strains in the pin-plate only; the fourth is a strain in the web only; the third comprises a strain in the web only, coupled with a movement of the end of the pin-plate, from whatever cause, with respect to the web; and finally, the sum of the first three strains represents the total strain in the web in the first 30 in. in front of the pin. In Fig. 9 the writer has collected the strains for Column No. 1. as recorded in Figs. 1, 2, 3, and 4, of Plate XXXVII, averaging the strains for the two ends, and plotting them in accordance with this interpretation.

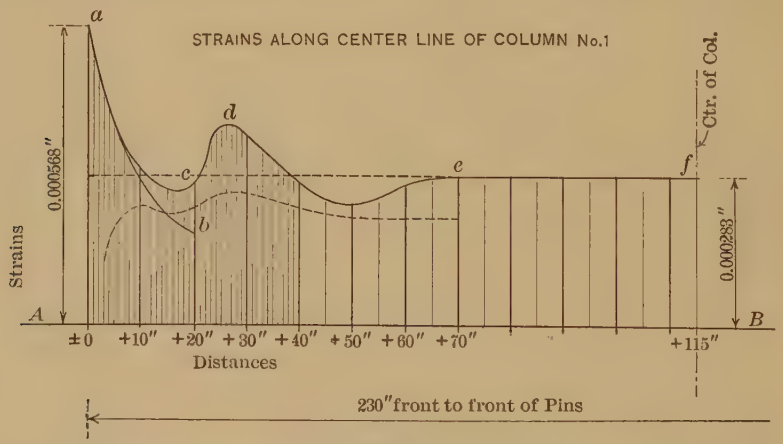


FIG. 9.

In this diagram distances along the center line of the column are given in inches along the line, *A-B*, while ordinates to this line represent strains. The lower line, *a b*, represents the outline of strains in the outer pin-plate, and the upper line, *a c d e f*, represents the outline of strains in the web-plate; all for one-half of the column, measuring from the front of the pin.

It will be recalled that the strains used in preparing this diagram are those due to a net load of 780 000 lb. applied to the column, giving unit stresses of 8 597 lb. per sq. in. in compression on the body of the member, 16 000 lb. per sq. in. in bearing on the diametral projection

of the pin, and, theoretically, 6 823 lb. per sq. in. shear on the rivets connecting the outer pin-plates to the web, this last being based on the usual assumption that the pin-plates take their share of the load in proportion to their thickness.

This diagram was derived and constructed as follows:

(1) The total strain in the pin-plate between 0 and +10 in., is made 0.0039 in., or the average of 0.0038 and 0.0040 in. given in Figs. 2 and 4, respectively, of Plate XXXVII, for the space just in front of the pin.

(2) The total strain in the pin-plate between +10 and +20 in. is made 0.00215 in., or the average of 0.0019 and 0.0024 in. given on Figs. 2 and 4, respectively, of Plate XXXVII, for the second space in front of the pin.

(3) The point, *a*, is common to the strain curves, *a b*, of the pin-plate, and *a c d e f* of the web, its ordinate being made 0.000568 in., to accord with Fig. 3, Plate XXXVII, and found by working backward from an assumed elastic limit in the material, as will be explained later.

(4) The total strain in the web, between 0 and +30 in., is made to equal 0.01015 in., or the average sum of the strains in the first three spaces in front of the pin, as recorded in Figs. 2 and 4, respectively, of Plate XXXVII.

(5) The total strain in the web, between +30 and +40 in., is made to equal 0.00315 in., or the average of the values, 0.0033 and 0.0030 in., as recorded in Figs. 2 and 4, Plate XXXVII.

(6) The total strain in the 150-in. length between +40 in. on one end of the column and +40 in. on the other end, is made to equal 0.0408 in., as given in Fig. 1, Plate XXXVII.

(7) The total strain in the web between +23.125 in. on one end of the column and +23.125 in. on the other end, is made to equal 0.0522 in., as given in Fig. 1 of Plate XXXVII, being the strain recorded for the length of 183.75 in.

(8) The line, *a c d e*, is made to become tangent to the horizontal by gradual curves, the ordinate to this horizontal measuring 0.000283 in., or the average strain per inch of the ten strains at the middle of the column recorded in Fig. 3, Plate XXXVII.

(9) The lines, *a b* and *a c d e f*, are assumed to be comparatively regular, without any sharp breaks in their length.

Thus constructed, the strain curves, *a b* and *a c d e f*, are determined almost as exactly as though each ordinate was determined separately. In some places slight variations from the curves as shown are possible, but not without corresponding and equivalent variations in the opposite direction somewhere else.

Fig. 9 represents only one column and one loading; and yet the writer, for the following reasons, attaches to it, as a whole, a

Mr. Worthington.

quantitative value which would not ordinarily be warranted under such conditions:

(a) While representing only one column in fact, it really represents the average of two entirely distinct test specimens—the two ends of that column—each of identical construction, the bulk of the material forming the corresponding parts in each being even rolled at the same time.

(b) Under not identical but actually the same loading, the measured strains at corresponding points on the two ends show only such reasonable variations as would be expected, and are practically a check on one another.

(c) The complete blending—without discrepancies or necessity for "fudging"—of the many different measurements and combinations of measurements used in constructing the diagram into a harmonious and reasonable whole, serves also to confirm the accuracy of the measurements themselves.

(d) The evidence generally of great care exercised by the author in making and recording the measurements of the test.

The perfect elasticity of all the strains used in this diagram is vouched for by the author in his statement that the strains "appeared and disappeared promptly when the loads were changed." The deliberateness with which the strains were measured is shown by the author's statement that "the maximum load, 800 000 lb., was applied several times before making the observations here recorded." Both these statements refer to the particular set of measurements used in preparing Fig. 9.

The dotted curve below the line, *a c d e f*, in Fig. 9, represents the corresponding average transverse strains, as taken from Figs. 3 and 4, Plate XXXVIII, and Fig. 1, Plate XXXIX, the ordinates to the line, *A-B*, representing the total strains measured in the width of the web. This curve is of interest in that it follows in a general way, as would be expected, the strain line, *a c d e f*. At the end of the column it dips in an opposite direction from the strain curve, owing, it is assumed, to the complicated distribution of the stresses there.

An interesting check on the diagram, Fig. 9, is found in Fig. 4. Scaling from that diagram the strains marked Nos. 1, 2, 3, and 4, at the first break in line No. 1, or at a pin pressure of about 23 500 lb., the resulting values of strains No. 1, No. 2, Nos. 1, 2, and 3, combined, and No. 4, are 0.0056, 0.0027, 0.0149, and 0.0049 in., respectively. Multiplying the corresponding values used in Fig. 9 by $\frac{23\,500}{16\,000}$, in order to make allowance for the increased load, these strains should be 0.00572, 0.00316, 0.0149, and 0.00463 in., respectively. It will be observed that these two sets of values, one set measured, and the other representing forecasted values based on strains in an entirely different

column, agree very closely, and, in so doing, serve as an excellent check on Fig. 9 generally, and on the elastic property of the strains involved in that diagram.

A similar attempt to check Fig. 9 by Fig. 5 was not as satisfactory. In fact, the stress-strain lines of this diagram are not at all consistent with Fig. 9, but the writer has elected to consider it inconclusive, rather than as invalidating this diagram.

Considering now Fig. 9 in its bearing on the actual column and on the records of the test generally: The most conspicuous feature of this diagram is the abrupt rise in the strain curves at $\pm$ 0, the point just in front of the pin. The unit strain, per inch of column length, is shown to increase from 0.000283 in. in the middle of the column, to 0.000568 in., at the end just in front of the pin, indicating that the corresponding stresses in the material are in like proportion. Applying this diagram to the stress-strain curve for one end of Column No. 1, as recorded in Fig. 3, this stress-strain curve reaches its elastic limit at a pin pressure of about 28 000 lb., corresponding with a compressive stress in the body of the column of 15 100 lb. per sq. in. According to Fig. 9, the maximum stress, occurring just in front of the pin, would then be

$$15\,100 \times \frac{0.000568}{0.000283} = 30\,300.$$

This value was assumed to be the probable elastic limit of the pin bearing, being the lowest elastic limit given on Plate XXXII for the web-plates, and, by working backward, fixed the value of 0.000568 in. for the ordinate at $\pm$ 0 in Fig. 9, as stated above. This being done, Fig. 9 is consistent throughout with the record for Column No. 1.

Owing to the acute angle made by the strain curves, *a b* and *a c d e f*, with the ordinate at $\pm$ 0, the actual quantitative value of the ordinate at $\pm$ 0, the point just in front of the pin, is not determinable with any great degree of accuracy, except within rather wide limits.

While the sharp upward turn of the strain curves at $\pm$ 0, the point just in front of the pin, is the most conspicuous feature of Fig. 9, the most important is the means this diagram gives of determining the quantitative value of the elastic slip that occurs between the web and the pin-plates, and the relative value of web and pin-plates in bearing on the pin. In Fig. 9 the irregular triangle, *a b c*, represents the difference between the sum of all compressive strains in the pin-plate from $\pm$ 0, the front of the pin, to + 20 in., the end of the pin-plate, and those in the web between the same points. The amount is found, by scaling the ordinates at intervals of 1 in., to be 0.000605 in. The meaning of this is that, by reason of the difference of compressive stresses in pin-plate and web, the end of the pin-plate will move forward from the pin farther by this amount than the corresponding

Mr. Worthington.

point on the web, provided, of course, there are no other influences acting to modify the result. By Fig. 1, Plate XXXVII, however, the actual measurements show that the end of the pin-plate moved an average of 0.00100 in. farther forward than the corresponding point on the web. The difference between these amounts, or 0.000395 in., represents a slip of the pin-plate in addition to the movement of 0.000605 in. due to elastic distortion under direct compression. Thus, then, about 60% of the movement is due to direct compression, and results from the lack of continuity in the pin-plates, while the remaining 40% is due to other causes. Owing to the complexity of these other causes, the writer does not believe further quantitative analysis of this remaining 40% is warranted, but thinks it is probably all due to elastic distortion or "give" in the rivets connecting the pin-plates to the web.

The practical result of these movements in the pin-plates is a reduction of their efficiency in bearing, as compared with the web; the movement due to lack of continuity in the pin-plates is probably not reducible to any extent, but applies to all pin-plates and similar details where a continuous and a non-continuous elastic member are coupled together, as by riveting, to transfer a load; the movement due to slip, being caused by the elasticity of the bond between the pin-plates and the web, is directly affected by changing the amount of that bond; thus, if the number of rivets connecting the pin-plates to the web were doubled, this movement would probably be halved.

In arriving at a quantitative value for the efficiency in bearing of these pin-plates in terms of the web bearing, the writer will use for illustration a load supported through the intermediary of two pairs of elastic members, all of equal length before the application of the load, but each pair having a different coefficient of elasticity in that length. The load will then distribute itself to the two pairs of supports in the inverse ratio of their coefficients of elasticity, it being assumed that the supports are symmetrical with respect to the load and parallel with its direction. The load, in the case at hand, is the pressure of 16 000 lb. per sq. in. exerted by the pin, and the supports are the web and pin-plates; the length of the supports before applying the load is 20 in., or the distance from the front of the pin to the end of the pin-plates. The total strain in the web caused by the load is represented in Fig. 9 by the area between the base line, *A-B*, and the portion, *a c*, of the strain curve, between the ordinates at $\pm$ 0 and at $+20$ in., and amounts to 0.006655 in. The total net strain in the outer pin-plate is equal to the total strain in the web thus found, less the amount, 0.001000 in., found by measurement to be the movement of the end of the pin-plate with respect to the same point on the web, or 0.005655 in. Letting p and p_1 represent the respective portions of the total load of 16 000 lb. carried by equal areas of the web and pin-plate; and letting E and E_1 represent

Mr. Worthington

the respective coefficients of elasticity in 20 in. of the web and pin-plate, there may be written the following equations:

$$p + p_1 = 16\,000 \quad \ldots\ldots (1)$$

$$\frac{p}{p_1} = \frac{E_1}{E} \quad \ldots\ldots (2)$$

$$\frac{0.006655 \text{ in.}}{20 \text{ in.}} = \frac{p}{E} \quad \ldots\ldots (3)$$

$$\frac{0.005655 \text{ in.}}{20 \text{ in.}} = \frac{p_1}{E_1} \quad \ldots\ldots (4)$$

Solving these equations, $p = 8.294$ and $p_1 = 7.706$; or, expressed differently, $p_1 = 0.922\ p$.

Thus the efficiency of the pin-plate in bearing on the pin is only 92.2% of the web, which means that, in determining the amount of pin bearing, that portion made up by the pin-plates should be increased some 8 to 10% in order to make allowance for the combined non-continuity of the pin-plates and the elastic distortion of the rivets connecting them to the webs.

This has a wide and important bearing on design generally. For example, in addition to all pin bearings it applies directly to all partial splices, such as those occurring in the cover-plates and angles of long plate girders. Here the deficiency of the splice would be double the deficiency of one-half, the pin bearing of the test representing only one-half of such a splice, and for the same proportion of continuous and non-continuous portions as used in making the above calculations, the efficiency of the splice would be only from 80 to 85% as compared area to area with the continuous portion of the flange.

It applies also, and with much greater force, the writer thinks, to the practice of counting as effective flange area, one-eighth of the area of spliced webs, and then attempting to splice the web so that it will be effective. The writer has long felt that this is bad designing, for the reason that while the web splice alone would offer the resistance in bending allotted to it, yet when considered as acting in conjunction with the much more rigid flange, it would not come into play to anything like its full value until the stress in the flange had reached a much greater value, thereby causing the girder as a whole to reach its elastic limit at a point much below the elastic limit of the material, just as the defective pin bearing of the tested columns brought down the elastic limit of the columns as wholes in the case at hand. The discrepancy between the continuous flange and the web splice is in this case aggravated by the fact that the flange is farther from the neutral axis than the rivets forming the web splice, so that the "give" in the rivets of the splice is of much greater importance than if they were in the same line with the flange.

Mr. Worthington.

Having considered the influences affecting the strain curves in Fig. 9, an attempt will now be made to find an approximate value for the working pressure on the pin which will correspond with the working compressive stress in the body of the column.

In a well-proportioned column, the unit stress at any point should not exceed that in the body of the column. From this it follows that in Fig. 9 no point of the strain curve should lie above the horizontal portion, *e f*, extended. The ordinate at ± 0 should then be equal to or less than the ordinate, 0.000283 in., of the horizontal portion, and it is desired to find the unit bearing pressure which will cause these ordinates to become equal.

As Columns Nos. 2 to 5 correspond more nearly with usual conditions as to the $\frac{l}{r}$ value, the writer will assume a typical column of the same length as Columns Nos. 2 to 5 in conjunction with the end strains illustrated for Column No. 1 in Fig. 9. As stated above, the working load for Columns Nos. 2 to 5 is 1 143 000 lb. The effective pin bearing, considering the web at full value and the pin-plates at 92.2%, would be 46.99 sq. in., giving an effective working pressure (as now constructed) of 24 325 lb. in pin bearing. In order that the ordinate at ± 0 shall equal 0.000283 in., this pressure should be,

$$24\,325 \times \frac{0.0002830 \text{ in.}}{0.0005680 \text{ in.}} = 12\,124,$$

or about the same value as the compressive stress in the body of the column, which was found above to be 12 700 lb. per sq. in.

This confirms Mr. Carpenter's suggestion as to allowable pin pressure, and goes farther, indicating that instead of the pin pressure not exceeding the base of the compression formula, as he suggests, it should not exceed the compressive stress itself.

This is in line, too, with the drift of opinion as to the proper bearing pressure on eye-bars, as gathered by the writer from the discussion of the paper by Theodore Cooper, M. Am. Soc. C. E., entitled "New Facts about Eye-Bars."* It is also in line with what one would expect. It seems inconsistent to use different values for compressive stress at different points in the same member, and the stress just in front of the pin is as surely compressive as is that in the body of the member.

Whether this conclusion should extend to rivets, to the extent of limiting their bearing value to 16 000 lb., the writer would not care to say positively, but it would seem to be inevitable unless the gripping effect of the rivet heads be given some value to warrant the higher bearing values of usual practice, and this he does not think good, because this gripping effect is too uncertain, and may not exist at all after a time.

* *Transactions*, Am. Soc. C. E., Vol. LVI, p. 411.

Mr. Worthington

Another important feature of Fig. 9 is the upward bulge in the strain curve, *a c d e f*, at *d*. The stress-reducing effect of the pin-plates is indicated by the dip in this curve at the point, *c*, but at *c* the pin-plates terminate, and thereupon the stress becomes greater again, reaching a maximum at *d*, and then diminishes until it equals that in the body of the column, about 39 in. from the front of the pin, thus indicating that, for this column, the pin-plates (or at any rate part of the pin-plates) should have been extended to this point to meet theoretical conditions; and then somewhat farther to make sure that the theoretical conditions would be met in all cases. In other words, it appears that the pin-plates, or part of the pin-plates should have a length, in this case, of about 45 in. from the front of the pin, or one and one-half times the width of the web.

The coefficient of elasticity of the column over the horizontal portion, *e f*, of the strain curve, *a c d e f*, in Fig. 9, is,

$$8\,597 \times \frac{1.000000}{0.000283} = 30\,380\,000,$$

whereas the coefficient of elasticity of the column in the whole length of 230 in. from front to front of pins would be,

$$8\,597 \times \frac{230}{0.0674} = 29\,340\,000,$$

where 0.0674 in. is the total strain, as represented by the area between the line, *A-B*, and the strain curve, *a c d e f*, both ends included.

The first found coefficient, 30 380 000, is the coefficient of elasticity of the column shaft, and would compare directly with the coefficient of elasticity which might be forecasted by averaging the values of *E*, given on Plate XXXII, for the component parts of this column, if the items on this plate were marked so that they might be distinguished in the column. As the record stands, it is not possible to average the values given for *E*, because this plate includes, in the total of eight test specimens, four that represent one angle each of 7.11 sq. in., two that represent one plate each of 10.94 sq. in., and two that represent one plate each of 20.625 sq. in., without any means of distinguishing the plates one from another. It would be interesting to have each of these plates identified, so that the probable value of *E* for the column could be forecasted, in order to compare it with the actual value as found above. The writer believes the restraining effect of the rivet heads on the strain would cause the actual coefficient of elasticity to be greater than the value forecasted from the component material.

The second coefficient found, 29 340 000, is not a true coefficient of elasticity at all, but represents the value which should be used as the coefficient of elasticity for this member when considered as part of an elastic structure. If, in Fig. 9, the area between the line, *A-B*, and the strain curve, *a c d e f*, should be equal to that between that line

Mr. Worthington.

and an extension of the horizontal portion, *e f*, to the front of the pin, then this coefficient would equal the first found coefficient. If, now, there were an excess of pin-plates, so that the portion, *a c d e*, of the strain curve were generally below the extension of the horizontal portion, *e f*, then this coefficient would be greater than that first found.

For ordinary structures, the coefficient of elasticity of the member does not enter into the process of designing; but, for very large structures, and for those like arches, where it is necessary to make use of the coefficient of elasticity, this is an important matter.

For Column No. 3, the equivalent of the second coefficient found above for Column No. 1 would be, applying Fig. 9 directly thereto and making allowance for the increased length,

$$8\,597 \times \frac{427 \text{ in.}}{0.12315 \text{ in.}} = 29\,810\,000,$$

which represents the coefficient of elasticity of the column from front to front of pins. The coefficient of elasticity recorded for this column from center to center of pins, as found by scaling from Fig. 7, is about 23 800 000. The writer can find no satisfactory reason for this wide discrepancy. The coefficient of elasticity in the 240-in. length agrees pretty closely with what one would expect, and with the other columns, which indicates that there must be a considerably greater rise in the ends of the strain curve of Fig. 9, as applied to this column, than that shown in the diagram as prepared for Column No. 1. Against this supposition there is the generally straight stem of the stress-strain line in Fig. 7 up to a point where the stress equals about 21 000 lb. per sq. in., when it breaks away from the straight line because of the fact that, when the stress in the body of the column has reached a compressive stress of 21 000 lb., the stress in the material just in front of the pin would be much greater, and probably at the elastic limit of the material; if the same ratio existed as that indicated by Fig. 9 for Column No. 1, the stress just in front of the pin would be

$$21\,000 \times \frac{0.000568}{0.000283} = 42\,150 \text{ lb.},$$

and at any such stress, of course, the material would have begun to fail. The writer is inclined to think there is some error in preparing or in plotting this stress-strain line on Fig. 7, as the discrepancy is too great to be compatible with the other phenomena recorded for this column. Whatever the cause, it will be noted that it does not affect the general elasticity of the column, as indicated by the practically straight stem of the stress-strain curve in Fig. 7.

In reference to the permanent sets recorded by the author: While material generally is erratic in behavior above the elastic limit, and a built-up structure like a column would probably be even more so, these permanent sets are of interest.

Mr. Worthington.

In the light of Fig. 9 it is evident that the greatest permanent set would be expected to occur just in front of the pin, and the next greatest at the apex, *d*, of the strain curve near point +30 in.; while there would probably be no permanent set in the pin-plate at point +20 in., and possibly none throughout the greater part of space from +10 to +20 in.

The permanent set in the web between points +10 and +20 in. would probably be much greater than that in the pin-plates in this same space.

The permanent sets recorded in Fig. 3, Plate XXXIX, for the east end of Column No. 2, bear out these conclusions completely, and the writer sees no reason to assume that the pin-plate "had moved bodily along the column," as assumed by the author, because all the apparent movement is readily accounted for by the expected difference in compressive strains in web and in pin-plate. By plotting these recorded strains in the same manner as for Fig. 9, it will be seen that there is little leeway for any "slip."

The other permanent sets recorded do not admit of definite conclusions, as they are all limited to the first three spaces in front of the pin, except those in Fig. 4, Plate XXXIX, where the set in the fourth space is given. Inasmuch as this set of 0.0180 in. is five times as great as that at the corresponding point at the opposite end of the same column, which latter is confirmed by an additional set recorded in a fifth space, the writer would be inclined to throw this set out, as being probably in some error.

Without going into detail further, the writer does not think there is any evidence in the record of tests that failure of the bond between pin-plates and web caused any appreciable permanent slip in the pin-plates.

An interesting feature of the stress-strain line No. 3, in Fig. 4, is the break in continuity of this line at a pin pressure of about 12 000 lb. This break being backward, indicates a hardening of the resistance represented by this line. Including, as it does, the slip of the pin-plates, it would appear to the writer that this break in continuity represents the coming into play of the rivets in bearing, the lower portion representing only the gripping effect of the rivet heads, and the upper representing the combined gripping and bearing effects of the rivets.

In conclusion, the deductions from these tests, in their bearing on design, may be summed up as follows:

(1) They confirm the usual relation between working stresses in tension and compression represented by the values of 16 000 and $16\,000 - 70\frac{l}{r}$, respectively, as being consistent with one another.

Mr. Worthington.

(2) They indicate that, for pin-plates, the efficiency in pin bearing should be taken at, say, 90%, as compared with the continuous webs.

(3) They indicate that, for partial splices, as of angles and cover-plates of girders, the efficiency of the splicing metal should be taken at, say, 80%, as compared with the metal in the continuous section; and that the practice of counting one-eighth of the area of spliced webs as effective flange section should be abandoned.

(4) They indicate that pin-plates, in part at any rate, should have a length of, say, one and one-half times the width of the member, measuring the length from the front of the pin.

(5) They indicate that the working pressure on pins should not exceed the working compressive stress in the body of the member, this working pressure having reference to the continuous webs, while that portion of the bearing area made up by the pin-plates should be increased further, as outlined in Conclusion (2).

(6) They indicate that the bearing value of rivets should also probably not exceed the value used for direct compression, or, say, 16 000 lb.

Although the writer has given definite figures in this discussion, it is to be understood that these are intended to be approximate only, as further experimenting would be necessary to establish them accurately.

Mr. Kunz.

F. C. Kunz, M. Am. Soc. C. E. (by letter).—Mr. Howard's excellent paper should be of more than passing interest to structural engineers, and the thanks of the Profession are due to him and to Mr. J. C. Bland who originated the tests.

The data presented prove once again that established theory needs revision from time to time in the light of new experience with full-sized members. Formulas are based on certain assumptions which, however, are often lost sight of, and the formulas are applied without limitations until reliable tests disclose the discrepancy.

At the suggestion of Mr. Bland, the writer was present in Watertown and Phœnixville, where the tests were made, aud he had the privilege of witnessing the extraordinary care with which they were conducted by Mr. Howard and the late Mr. Buchanan.

Although no definite conclusions can be drawn from these isolated tests they furnish one more proof that latticed columns do not develop the same buckling strength as solid columns or those with solid web diaphragms or cover-plates. This fact disclosed by recent failures and test results has perhaps long been realized by engineers, but it is only within the last few years that more theoretical consideration has been given to the subject.

It is the writer's intention to point out the probable reasons for the discrepancy between the test results of latticed columns and the results of the column formulas which have been derived by Messrs. T. H

Mr. Kunz.

Johnson and L. v. Tetmajer from tests, with more or less solid columns, made by Messrs. Tetmajer, Bauschinger, Hodgkinson, Christie, and others. There are two principal reasons for this discrepancy:

First.—The ribs which make up the latticed columns form themselves independent columns of the length l_1 between two points of connection of the latticing. Generally it is assumed that the strength of the column as a whole is fully developed if the ratio, $\frac{l_1}{r_1}$, of the rib as an individual column is not greater than the ratio, $\frac{l}{r}$, of the whole column, both ratios being taken in a plane parallel to the latticing. This is not correct, however, as is easily understood when it is considered that the failure of the column is the result of excessive fiber stresses due to bending. It is evident that the bending stresses in the rib as an individual column and as a part of the whole column, are added, thus causing the rib and, therefore, the column as a whole, to fail under a smaller load than in the case of a solid column. The fact that the columns failed by the local buckling of a rib seems to prove this reasoning. Analytically, it can be proven as follows:

In the straight-line formula, for the average buckling strength of solid columns, $s = k - c\frac{l}{r}$, the value of $c\frac{l}{r}$ may be regarded as the reduction of the ultimate strength, k, due to the bending stresses. The bending stress in the outer fiber of a latticed column is, therefore, $c\frac{l_1}{r_1} + c\frac{l}{r}$; for the rib, the value, $c\frac{l}{r}$, represents practically a uniform stress which may be assumed to be caused by a load acting in the axis of the rib. The total axial unit stress in the rib is then $s + c\frac{l}{r}$, which must be equal to the buckling strength of the rib, $k - c\frac{l_1}{r_1}$; the buckling strength of the column as a whole is, therefore,

$$s = k - c\frac{l}{r} - c\frac{l_1}{r_1} = k - c\left(\frac{l}{r} + \frac{l_1}{r_1}\right) \quad \ldots\ldots\ldots\ldots (1)$$

The buckling length, l_1, is usually the distance between centers of connections of the lattice bars, since this may be the distance between points of contraflexure of the elastic line of the rib.

If the permissible buckling stress is $16\,000 - 70\frac{l}{r}$ for a solid column, the permissible value for a latticed column in the plane of the latticing, if, for example, $\frac{l}{r} = 50$ and $\frac{l}{r} = 40$, is, therefore, only $16\,000 - 70 \times 90 = 9\,700$ lb. per sq. in. In most cases, $c\frac{l_1}{r_1}$ is small; in some built compression members, however, it is quite considerable.

Mr. Kunz. In Column No. 5 tested by Mr. Howard, we have, at right angles to the latticing, $\frac{l}{r} = 47$, and parallel thereto, $\frac{l}{r} = 46$ and $\frac{l_1}{r_1} = \frac{14}{1.4} = 10$. According to Tetmajer $c = 160$, and the reduction of the buckling strength is, therefore $(46 + 10)\ 160 = 9\,000$ lb. per sq. in. parallel to the latticing, as against 7 500 lb. parallel to the rib, a difference of 1 500 lb. per sq. in.

Second.—The second principal reason for the reduction of the buckling strength of latticed columns is the fact that the deformation of the lattice bars may have a considerable influence on the lateral deflection of the column as a whole, thereby increasing the bending moment and the bending stresses. It has been generally assumed that the shearing stresses have a negligible influence on the deflection. This, as can easily be proven, is correct for solid columns or for those with solid diaphragms which are in any reasonable proportion to the cross-section of the ribs, but it is not always negligible in latticed columns. This is analogous to the fact that the deflection due to the shearing stresses in a solid plate girder is generally small, while that due to the deformation of the web members in a truss is often considerable. Now, if it is considered that with increasing load a deflection of the column, if once started, increases much more rapidly than the load, it can easily be understood that the deformation of the lattice bars may considerably reduce the buckling strength. This can be expressed analytically as follows:

For long columns, $\left(\frac{l}{r} > 100\right)$, the buckling strength can be expressed by Euler's formula,

$$s_o = \pi^2 E \left(\frac{r}{l}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

In the derivation of this formula the deflection due to the shearing stresses has been neglected. If it is considered, the buckling strength becomes,

$$s = \frac{s_o}{1+n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where $n = \frac{y_s}{y_b}$, that is, the ratio of the deflection, y_s, due to the shearing stresses to that due to the bending stresses. For latticed columns, the ratio, n, can be expressed approximately by the formula,

$$n = \alpha \left(\frac{r}{l}\right)^2, \text{ where } \alpha = 9.6\ \frac{A\ t^3}{A_s f\ h^2} \quad \ldots\ldots\ldots\ldots\ldots (4)$$

and A = area of the column section,

A_s = area of section of all lattice bars cut by a transverse section through the column,

h = distance between centers of gravity of the two outer ribs,

f = panel length of latticing, and,

$t = \sqrt{f^2 + h^2}$. (Fig. 10.)

Mr. Kunz.

The ratio, n, is independent of the modulus of elasticity, and applies therefore approximately also if the stresses exceed the elastic limit; in other words, Formulas 3 and 4 may also be applied to short columns, $\left(\frac{l}{r} < 100\right)$, provided s_o represents the buckling strength of solid columns.

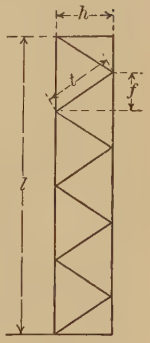


FIG. 10.

Assuming, for structural steel, Tetmajer's formula,

$$s_o = 45\,000 - 160\,\frac{l}{r} \quad \ldots\ldots\ldots (5)$$

and plotting the results of Formulas 3 and 4 for a certain value of α in a diagram (Fig. 11), we obtain a curve, $o\ f\ b_1$, which shows that the effect of the shearing stresses and, therefore, the reduction in the buckling strength, increases with decreasing ratio, $\frac{l}{r}$. On the other hand, with decreasing $\frac{l}{r}$, the buckling strength of the column as a whole must approach the buckling strength of a single rib with the length, l, that is, the stress corresponding to the ratio, $\frac{l}{r_1}$, if r_1 is the radius of gyration of the rib parallel to the latticing. This stress, for a given ratio, $\frac{r}{r_1}$, can be represented by a curve, $a\ e\ b_2$. The buckling strength of the latticed column as a whole will, therefore, probably be represented by a curve of the shape, $a\ d\ b_1$. If we assume it to be a parabola with a horizontal tangent at b_1 $\left(\text{for } \frac{l}{r} = 100\right)$, the results of this calculation agree reasonably well with the few test results of columns which failed as a whole parallel to the latticing.

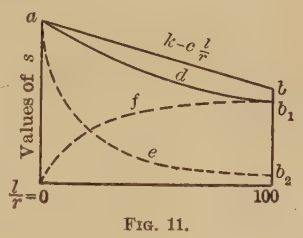


FIG. 11.

If we denote the value of n for $\frac{l}{r} = 100$ with p, Formula 4 becomes,

$$p = \frac{\alpha}{10\,000} = 0.00096\,\frac{A\,t^3}{A_s\,f\,h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

For any value of $\frac{l}{r}$ smaller than 100, we then get,

$$s = \left(45\,000 - \frac{29\,000}{1+p}\right)\left(\frac{100 - \frac{l}{r}}{100}\right)^2 + \frac{29\,000}{1+p} \quad \ldots\ldots\ldots (7)$$

Mr. Kunz.

Applied to the column tested by Mr. Howard, we find $p = 0.14$, and the buckling strength, according to Formula 7, is $s = 31\,100$. Deducting from this the stress, $c \frac{l_1}{r_1} = 160 \times 10 = 1\,600$, we get $s = 29\,500$, which compares favorably with the 30 500 lb. per sq. in. at which the column failed in the test, while the buckling strength of a solid column of the same $\frac{l}{r}$ would be $s = 45\,000 - 160 \times 46 = 37\,700$. In other columns tested recently the reduction is much greater.

In view of the foregoing, it would seem that the usual permissible buckling stresses are too high for latticed columns, and the question arises as to whether lower unit stresses should be used or whether provision should be made in each case for the above-mentioned reductions in the buckling strength. The writer is of the opinion that the unit stress of $16\,000 - 70 \frac{l}{r}$, for static loads or for live loads plus impact, is perfectly safe provided the columns are properly designed. As a rule it is possible to make the $\frac{l}{r}$ parallel to the latticing so much greater than in the other direction that, even if the above reductions are considered, the column would fail at right angles to the latticing. Further, the column can usually be designed so that the above reductions become almost negligible. In any case the latticing should be dimensioned to resist safely the shearing stresses which may be developed in its plane. If this is not practicable or economical, then cover-plates or a full web diaphragm should be used, preferably in addition to the latticing, if the ribs are very wide. Lower unit stresses, while undoubtedly necessary for poorly designed columns, would rather encourage the use of insufficient details and cause a waste of material in properly designed columns.

It may be of interest to note in this connection that in the latest Prussian Government Regulations (1910), the permissible unit stresses have been increased and are considerably higher than those in use in the United States. The specified unit stress in compression members of trusses for dead and static live load is either 17 100 lb. per sq. in., or one-quarter of the buckling stress as derived from Euler's formula, whichever gives the greater section. There is, therefore, no reduction of the 17 100 for values of $\frac{l}{r}$ up to about 21 (for $\frac{l}{r} = 21$, the formula, $16\,000 - 70 \frac{l}{r}$, gives only 14 500). If wind is considered, the 17 100 may even be increased in roof trusses to 22 800. For steel columns the permissible stress is either 17 100, or one-fifth of Euler's buckling stress.

Mr. Kunz.

The danger does not lie in these high unit stresses, but rather in the use of Euler's formula. Since in this formula $\frac{l}{r}$ is in the second power, the influence of the selected buckling length l on the permissible stress is much greater than in the straight-line formula where l is in the first power only. This was the main reason for the failure of a large gas tank (capacity, 7 000 000 cu. ft.) in Hamburg, Germany, which occurred on December 7th, 1909, killing 20 people. The failure was due to the buckling of a column for which only 0.7 of the length between connections was used as buckling length instead of the full length as specified in the regulations. This, together with a somewhat underestimated load and the weak details (tie-plates only being used), reduced the intended factor of safety of 5 to less than 2. The $\frac{l}{r}$ was 93 (taking l as the full length between connections), and the unit stress under the greatest load was 17 800 lb. per sq. in., while only 7 100 lb. is permissible according to the above regulations.

More reliable data for the strength of latticed columns can only be obtained from a great number of comparative tests, and it is to be hoped that the two powerful testing machines now being constructed will furnish such opportunity.

Mr. Howard.

James E. Howard, Esq. (by letter).—The writer desires to express his thanks for the cordial manner in which the results of the tests of these interesting columns, made available through the courtesy of Mr. Bland, have been received. The discussion has brought out ideas and suggestions of great value which will materially aid further experimental work. The opportunity will be taken to add a few remarks prompted by those suggestions, although, because of lack of information, it is not possible to reply to all the ideas which have been presented.

First, however, as to the value of tensile tests of the component parts of a column as an index of its compressive behavior when assembled: Provided the tensile test has a value in this connection, it would seem to be on account of this value being the same in tension as in compression, or that a definite ratio existed between the two.

Experiments have shown that the elastic limits are substantially the same in tension as in compression, under conditions of treatment similar to that which is usually given structural steels, and, further, that peculiarities in the stress-strain curves, in the vicinity of the elastic limit, are also similar in tension and compression. That is, steels which in tension show a jog in the stress-strain curve at the elastic limit, a zone in which the strains increase without increase of stress or even increase under diminished load, have corresponding jogs when tested in compression.

Mr. Howard.

Some tests will be quoted, made on specimens differing widely in their elastic limits and tested on widely different occasions, in which the elastic limits were determined for the same metal in tension and in compression:

Description.				Elastic limit, in pounds per square inch, in: Tension.	Compression.
Steel	forging,	tested	1888.....	62 000	65 000
"	"	"	1887.....	48 000	47 000
"	"	"	1902.....	35 000	35 000

Each of these steels had a jog immediately following the elastic limit, and in that first quoted, the load fell from 65 000 lb. per sq. in. compression to 62 000 lb., and for a time continued to yield under that reduced load.

In steels which display a jog there can be no doubt concerning the position of the elastic limit, as it is so sharply defined. The plates and angles which were used in the fabrication of these five columns showed these characteristic jogs.

This circumstance doubtless fixed the ultimate resistance of each column at substantially the elastic limit of the component member which had the minimum value. On account of the rapid yielding at the elastic limit, or the temporary drop in resistance while rapid yielding is in progress, a close approach to that limit signals a close approach to the ultimate strength of the column. For this reason it was believed that the strength of Column No. 5 represented the ultimate strength of the other four columns of the series.

In the harder steels, without jogs at the elastic limits, so exact a prediction of the ultimate resistance would not be warranted, and in the case of wrought iron there would be an opportunity of attaining a little higher resistance, on account of the usual absence of a jog.

In all large columns—because in large columns those unavoidable imperfections of material and workmanship appear to have relatively less influence than in small columns—it is believed that an ultimate resistance coincident with the elastic limit of the individual members should be attained, while, from the ease with which local buckling would then take place, it is believed that no higher resistance substantially will be realized nor should be expected.

It is important that this last statement should be verified, as far as it may be, since the ultimate strength of large compression members is likely to continue to be a matter of conjecture, lacking direct experimental evidence on account of the strength of columns found in current architectural and engineering examples greatly exceeding the capacity of present or contemplated testing machines. The correlation of results of tests, conducted after the manner of the five columns herein reported, on comparatively small compression members in the testing machine, with reliable strain measurements on dead-load stresses on members in actual structures, is the only method which,

Mr. Howard.

to the writer, appears to be adequate to furnish data for judging of the efficiency and strength of large members of current design and manufacture.

Concerning the modulus of elasticity, careful determinations indicate a value in the vicinity of 29 500 000 to 30 000 000 lb. per sq. in., for steels of the usual range in carbon content. Until permanent sets are developed, each component part of a properly designed and fabricated column should sustain that part of the total load which is proportional to the sectional area of the part in question. Such a condition, strictly considered, can hardly prevail except under quite low unit stresses.

The presence of permanent sets will be felt under loads below the elastic limit of the parts as a whole, due to internal strains of a more or less local character, and the distribution of the stresses may be disturbed thereby; but some of the features which induce internal strains still tend to keep the loads axial on the column. Internal strains in the plates and angles, resulting from rapid cooling after rolling, would generally be so disposed as to keep the loads central, if originally so. The effect of sudden cooling from a temperature less than a low red heat is adequate to introduce internal strains corresponding to a stress of 30 000 lb. per sq. in. Angles of unequal legs might be under different cooling strains, and likewise the webs and flanges of channels; but, withal, the parts overstrained in a column might be successively developed and yet symmetrically maintained, and the column remain straight as loads are advanced.

Cold straightening or local indentations would hardly be so disposed as to leave their effects evenly distributed over the cross-section of the column. Steel overstrained cold no longer retains its equality of elastic limits in tension and compression, although time may effect a modification. It has been shown experimentally that light hammer blows, indenting the steel, has induced internal strains in a zone of metal $\frac{1}{8}$ in. thick, sufficient to expand measurably the bore of a cylinder having walls $3\frac{1}{2}$ in. thick.

The contractile force of rivets in cooling has been referred to, and such contraction induces internal strains under the heads and in the immediate vicinity. Generally, such strains may be well distributed, but, if the plates drawn together were not originally flat, the contraction of the rivets last driven might loosen those first driven by reason of further flattening of the plates.

In the test of a column, the first evidence of locally passing the elastic limit of the metal is found where these local strains exist and are of the same sign as the stress on the column. In the present tests, lines of scale first started off the body of the columns in the vicinity of the rivets, extending from rivet to rivet diagonally and also at right angles to the axis of the column. It is understood, of course, that scaling shows where the elastic limit is being exceeded.

Mr. Howard.

In these columns the earliest scaling and development of permanent sets naturally occurred in front of the pins, where the compressive stress was nearly twice that on the full section of the column. This greater unit stress accounts for the failure of two of the columns by buckling in front of the pins. The reinforcement of the most strained metal directly in front of the pins by the successively less strained metal beyond enabled a higher unit stress to be sustained in their vicinity than would have been possible over the full cross-section of the column.

Concerning strains in lattice bars, practically none have been found in columns in the testing machine. Observations were made, in another series of tests, on all the bars on one side of a column, some thirty in number. With a stress of 20 000 lb. per sq. in. on the channels, no measurable strains were found on the majority of the lattice bars; in some there was a compressive strain of $\frac{1}{10\,000}$ or $\frac{2}{10\,000}$ in., on gauged lengths of 6 in.

With pronounced deflection or buckling of the column, doubtless strains are introduced, but in the testing machine this occurs at a later stage, in fact, it is usually after the maximum load has been passed and the column is failing under diminished stresses. The customary photographs of columns after failure commonly show their appearance some time after passing their ultimate resistance. At the immediate time of attaining the maximum stress many columns show less distortion than would be visible in a photograph, although, in some cases, the maximum load may be sustained while deflection slowly goes on until the column is a considerable part of a diameter out of line.

To strengthen a column against loads beyond the elastic limit of the metal is to strengthen it against cold flow, and that appears to be impracticable.

Thin sections may be more seriously affected by internal strains than thicker sections, if extremes are compared, and buckle earlier by reason of the initial state of the metal. The higher elastic limit looked for in the thin section would, in itself, give it an advantage over the thick section, and so one practical consideration will be set against another.

It is desired to make a correction in regard to the stress-strain diagrams representing the plates and angles. They represent inspection work done, and were erroneously attributed to the Pittsburg Testing Laboratory. They were introduced to show the general behavior of the steels at the elastic limits, which displayed jogs, and to direct attention to the importance of the elastic limit rather than to the reported yield point. Obviously, the moduli of elasticity reported should not be accepted. The test pieces were probably unsuited for that purpose.

MEMOIRS OF DECEASED MEMBERS

LEFFERT LEFFERTS BUCK, M. Am. Soc. C. E.*

DIED JULY 17TH, 1909.

Leffert Lefferts Buck, third son of Lemuel and Elizabeth (Baldridge) Buck, was born on February 5th, 1837, in Canton, St. Lawrence Co., N. Y. He was descended from sturdy New England stock, his grandfather, some of whose immediate family attained considerable prominence in the Revolutionary War, having moved from Connecticut early in the 19th Century.

As a boy, Mr. Buck evinced that keen ambition to accomplish extraordinary results which in after life so strongly characterized him, and, in his home and school duties, developed the ingenuity and manual efficiency, which ultimately became remarkable. He then, as always, eagerly sought opportunity to cope with difficulties and to excel in feats of mind, skill, or strength.

He entered the first class of the college which afterward became St. Lawrence University, but had hardly completed the college course when the Civil War broke out, and he at once determined to enter the Army. His father, who was a close friend of the Governor of New York, wished to secure a commission for him, which he could doubtless have done; but Mr. Buck, with that characteristic regard for fitness which he applied to himself even more rigidly than to others, refused it because of his lack of military training, and in July, 1861, enlisted as a private in Company A, 60th New York Infantry. This was one of the most serious mistakes of his life, for, despite his unwillingness to accept the fact without test, he possessed an innate military ability that far outweighed his lack of previous training, and would doubtless have attained high rank had he entered the Army as a commissioned officer instead of as a private. However, he rose through the ranks of Corporal, Sergeant, Lieutenant, and Captain, until mustered out at the close of the war with the brevet rank of Major. He participated in some of the most arduous campaigns of the war, including the battles of Antietam, Chancellorsville, Gettysburg, Lookout Mountain, Ringgold, Resaca, Kennesaw Mountain, Peachtree Creek, and Atlanta, and also in Sherman's March to the Sea. In the famous "Fight Above the Clouds" at Lookout Mountain, it was he who finally carried the Union Flag to the summit, after two color-bearers had been shot down under it.

His indomitable will and contemptuous disregard for the restraining power of wounds and disease, were shown at Antietam where, after

* Memoir prepared by R. S. Buck, M. Am. Soc. C. E.

being shot through both legs, he kept afoot with his command for the rest of the day, and only after a night on the cold ground and the hopeless stiffening of his wounded legs made further walking impossible, could he be forced into the hospital. At another time, when recovering from a severe attack of typhoid, and still hardly able to walk, he learned that an important engagement was imminent. The surgeon refused to release him from the hospital, and, to aid in securing observance of his orders, put his patient's uniform out of his reach. However, Mr. Buck managed somehow to get his sword, and in his hospital clothes showed up with his command. He was again wounded at Resaca.

While Mr. Buck liked to talk over events of the war, he rarely spoke of those incidents wherein his personal participation was entitled to distinction. He always warmly esteemed and liked to meet those who faced him on the other side, and often expressed the wish that the soldiers of the North and South might some day fight side by side against a common enemy. While deploring many of the results of war he often said that he thought war preferable to too much peace, as he felt that too much peace bred indolence, selfishness, and avarice, while war developed the nobler traits of men.

While favoring an intelligent and fair pension system, Mr. Buck always greatly deplored what he considered a tendency to capitalize patriotism into an indiscriminate distribution of pensions for political purposes, and always refused to consider a pension for himself, although his wounds and health fully entitled him to it under the law.

He earnestly desired to remain in the regular Army at the close of the war, but decided not to do so because he lacked West Point training and was too old to enter the Military Academy. Immediately on being mustered out, he entered the Rensselaer Polytechnic Institute of Troy, N. Y., where he took the prescribed four years' course in Civil Engineering in three years, and was graduated in 1868.

For about three years after graduating, he served as Assistant Engineer in the Croton Aqueduct Department of New York City. He then spent two years in Peru, where he erected the first Verrugas Viaduct on the Lima and Oroya Railroad, and assisted in other work of construction on that road. The first Verrugas Viaduct was destroyed by flood some years later, and he designed and planned the erection of another, which is still in use, although most of the bridges on the same line have been replaced to meet the growing demands of traffic.

On his return from Peru, in 1873, Mr. Buck supervised the manufacture of the material for the bridge at Louisiana, Mo., one of the first large railroad bridges across the Mississippi River.

In 1875 he returned to Peru for about two years in charge of bridge erection.

In 1877 he became connected with the Railroad Suspension Bridge at Niagara Falls, which had been built in 1850 by the late John A. Roebling, M. Am. Soc. C. E. He reinforced the anchorages of the bridge, which had been badly overstrained, and replaced a large number of cable wires which had become seriously corroded about the anchorage shoes.

In 1880 he replaced the wooden suspended superstructure with one of steel, and in 1885 he replaced with steel towers the original stone towers, which were deteriorating under the combined influence of frost and motion due to moving loads. All this work of removal was accomplished without interruption of traffic, and was the most difficult, delicate, and daring piece of bridgework ever undertaken. This statement is made with full appreciation of the many great accomplishments since that time, but also with appreciation of the lack of precedent to guide and the complete dependence on his own ingenuity that faced the engineer in that work. Several friends, whose opinions were entitled to very serious consideration, urged him to suspend travel while doing this work, but he never once wavered in his determination to carry out the work as he had planned it.

In 1881, the Norman Medal of the American Society of Civil Engineers was awarded him for his paper on "The Reinforcement of the Anchorage and Renewal of the Suspended Superstructure of the Niagara Railroad Suspension Bridge."*

As finally reconstructed, the Niagara Railway Suspension Bridge served until 1896, when it was replaced by the present massive, two-hinged, spandrel braced, steel arch, which was designed by, manufactured, and erected under the direction of Mr. Buck. This arch was erected on the same center line as the Suspension Bridge, and the latter was taken down without interruption of traffic.

Associated with G. W. McNulty, M. Am. Soc. C. E., in 1888-89, Mr. Buck rebuilt the Niagara Falls and Clifton Bridge, a highway suspension bridge just below the Falls, which was originally built by Mr. Samuel Keefer, in 1868. In 1897, in order to provide capacity for trolley cars as well as for the increasing crowds of visitors, the Niagara Falls and Clifton Suspension Bridge was replaced by the longest steel arch span in the world, which was designed by and constructed under the direction of Mr. Buck. This work required maintenance of traffic during construction; however, in order to save time, expense, and risk, traffic was suspended for a few days when the arch was closed.

For his paper on this bridge, the Institution of Civil Engineers awarded him the Telford Premium in 1901.

Mr. Buck was Consulting Engineer of the Lewiston and Queenston Suspension Bridge across the Niagara River, built in 1899. He designed and constructed the Driving Park Avenue Bridge, and the

* *Transactions*, Am. Soc. C. E,, Vol. X, p. 195.

Platte Street Bridge, in Rochester, N. Y. The former was one of the early long-span spandrel braced arches constructed in the United States. He also designed and erected a number of bridges on the Northern Pacific Railroad during the extension of that road to the Pacific Coast.

By invitation of the Government Engineers, he made designs for the Connecticut Avenue Bridge and the Memorial Bridge across the Potomac at Washington.

His greatest work, at least in point of magnitude, is the Williamsburg Bridge across the East River between New York and Brooklyn. He was Chief Engineer of this bridge from the time of its inception in 1895 until its completion in 1903. After this he was retained as Consulting Engineer in the Department of Bridges in connection with this and other bridges, until a short time before his death, when he resigned on account of failing health.

For several years he represented Peru and Ecuador on the International Railway Commission, which planned a series of railroads connecting the United States with South America.

On June 4th, 1902, Mr. Buck married Miss Mira Rebecca Gould, of Paducah, Ky., who survives him.

Leffert Lefferts Buck will always stand forth as one of the heroic figures in the history of Engineering. He was resourceful, self-reliant, bold and aggressive in action, but as little disposed to force his ideas upon others as he was to let others force their ideas upon him, or force him from his own. He reached conclusions promptly and decisively, depended firmly on his own judgment, and was often resentful of divergent outside influences and opinions. He brought into his work a practical mechanical knowledge and a singleness of purpose which, while they sometimes restricted his vision of controlling necessities and ultimate broader aims, always placed on his work the stamp of originality and ability, often of a genius striding well ahead of his time.

His personal character was striking, and compelled admiration. In him the Puritan strain was strongly developed. Spartan-like, rugged, uncompromising always, and often showing a tenacity in the face of what to others appeared overwhelmingly adverse proofs, there was in his thoughts and actions a simple, earnest honesty, and an indomitable and unselfish love of right as he saw it. Below his severe, sometimes forbidding, exterior, was a loyalty, a generosity, a tenderness almost feminine.

Few men have steered a more direct course in life, wavered less in their estimates of right and wrong, or made less effort to win friendship and applause; and yet few men have had a greater number of devoted and loyal friends, or received more spontaneous and generous applause.

Mr. Buck was a Member of the Institution of Civil Engineers of England, the Loyal Legion of the United States, the Century Association, the Engineers' Club of New York, the Hamilton Club of Brooklyn, the Army and Navy Club, and the St. Lawrence County Society.

He was elected a Member of the American Society of Civil Engineers on February 3d, 1875, and served as a Director from 1892 to 1894.

GEORGE BOWERS CALDWELL, M. Am. Soc. C. E.*

DIED MARCH 31ST, 1911.

George Bowers Caldwell, son of Lucia Morgan and Lewis H. Caldwell, was born in Lowell, Mass., on April 11th, 1863, where he received his early education in the public schools. At the age of seventeen, he became an apprentice of the Lawrence Manufacturing Company, of Lowell, serving his time in the shops and drafting room. At the end of his four years' apprenticeship he remained with that company, as Assistant to the Master Mechanic, until 1889, when he accepted the position of Master Mechanic of the Washington Mills Company, of Lawrence, Mass., where, for four years, he was in charge of the drafting room, as well as the construction and repair work.

In 1893 Mr. Caldwell entered the employ of Westinghouse, Church, Kerr, and Company, and retained his connection with this firm until his death. During this period he was in responsible charge of designing and constructing many works of engineering interest, among which were the following:

The mechanical and electrical features of the South Terminal Station, at Boston, Mass.; the Pittsburg Terminal of the Pittsburg and Lake Erie Railroad; the railroad shops of the Pittsburg and Lake Erie Railroad at McKees Rocks; and the construction of the Kingsbridge Power Station of the Third Avenue Railroad. He also supervised the design and installation of the electrification of the Long Island Railroad, including the large power-house at Long Island City. He was in charge of the design and construction of the mechanical and electrical features of the Pennsylvania Terminal Station, in New York City, but was obliged to relinquish it just before it was completed. In all his work he showed engineering qualifications of a high degree.

In 1889 Mr. Caldwell married Mary E. Fraser of Lowell, Mass., who, with four children, survives him.

After the completion of his work in connection with the South Terminal Station, in 1899, Mr. Caldwell moved to Yonkers, N. Y., where he resided until his death.

He took an active interest in civic and social affairs, and was well and widely known in the neighborhood of his home, being an active worker and Trustee in the Baptist Church of the Redeemer and a Charter Member of the Nappeckamack Club, of Yonkers. He was also a Member of the American Society of Mechanical Engineers, of the Grecian Lodge, A. F. & A. M., and of the Mt. Sinai R. A. Chapter, of Lawrence, Mass.

* Memoir prepared by George B. Francis, M. Am. Soc. C. E.

Mr. Caldwell was a man of unwavering integrity and of great energy, inspiring perfect confidence and respect in all those with whom he dealt. He was not only a loving, but a wise parent, and his death leaves a void, not only in the hearts of his family, but in the hearts of those for whom he worked and who worked for him.

Mr. Caldwell was elected a Member of the American Society of Civil Engineers on April 1st, 1908.

FRANK LESLIE DAVIS, M. Am. Soc. C. E.*

DIED JUNE 9TH, 1909.

Frank Leslie Davis, son of David F. and Louisa Davis, was born at Wellsville, Ohio, on December 2d, 1858, and was reared on a farm. He taught school in Warren County, Iowa, and in 1884 was graduated from Simpson College. Later, he spent one school year taking special engineering work at Kansas State University.

Mr. Davis commenced his professional career with the Chicago, Rock Island and Pacific Railroad in 1887, at Wichita, Kans., where he had charge of forty miles of new construction work. He devoted the following ten years almost entirely to railroad engineering, during which time he was employed in various capacities on construction and location work by the Chicago, Rock Island and Pacific, the Southern Pacific, the Northern Pacific, and the Great Northern Railroads.

In 1897-98 Mr. Davis was City Engineer of Tacoma, Wash. In 1903-04 he was Assistant Engineer on the construction of the Puget Sound Power Company's plant near Tacoma. In 1904 he was again elected City Engineer of Tacoma, and continued to occupy this position until his death.

On October 14th, 1891, Mr. Davis was married, at Knoxville, Iowa, to Miss Alice Scoles, daughter of the Rev. J. J. D. Scoles, with whom he had become acquainted while at Simpson College. Mrs. Davis and a daughter, Miss Leslie S. Davis, survive him.

Mr. Davis was a prominent member of the Masonic Fraternity, holding the 32d degree. He was also a Shriner, a member of the Patrol of Afifi, and Master of A. F. & A. M. No. 22, of Tacoma, Wash.

He was a man of most exemplary habits, of strong convictions, and of aggressive initiative. As an engineer Mr. Davis contributed his full share to the upbuilding of the West, and, as a citizen, to the welfare of the community in which he spent his later years. The affectionate esteem in which he was held by his friends, by business and fraternal associates, and by his fellow-citizens generally, was most strikingly shown in the wide manifestation of sorrow and of sympathy for his family on the occasion of his death.

Mr. Davis was elected a Member of the American Society of Civil Engineers on September 6th, 1905.

* Memoir prepared by Arthur L. Adams, M. Am. Soc. C. E.

JOSEPH PALMER FRIZELL, M. Am. Soc. C. E.*

DIED MAY 4TH, 1910.

Joseph Palmer Frizell was born at Barford, Quebec, Canada, on March 13th, 1832. His parents, Oliver and Mary Beach Frizell, were natives of Vermont and belonged to good New England stock. His early education was received at Brownington, Vt., and Richmond, Canada. His teachers, discovering his aptitude in mathematics, taught him algebra and geometry, advising that he fit himself for Civil Engineering.

At the age of eighteen Mr. Frizell went to Manchester, N. H., where he was employed in a cotton mill until 1854, when he entered the City Engineer's office as an assistant. He devoted all his leisure time, both day and evening, to the study of his profession, and remained with the City Engineer until the close of 1856.

In 1857 he entered the office of the Locks and Canals Company, controlling the water power on the Merrimac River at Lowell, Mass., under the charge of the late James B. Francis, Past-President, Am. Soc. C. E., the eminent hydraulic engineer. Mr. Frizell was Assistant Engineer until 1861, when he left on account of the Civil War. He returned to this work in 1866, and remained until 1867. While in the Locks and Canals office he assisted Mr. Francis in the preparation of his treatise entitled "Lowell Hydraulic Experiments." In the preface of the edition of 1868, the author acknowledges his obligation to various persons, and also "to Mr. Joseph P. Frizell, now of Davenport, Iowa, to whom he is indebted for assistance in some points involving the higher mathematics."

In 1861, on the breaking out of the Civil War, he entered the United States Government Service as Assistant Civil Engineer, and was engaged until 1865 on the construction of Government fortifications along the Gulf Coast. He was stationed at Ship Island for 3 years, and at Davenport, Iowa, for 2½ years.

In 1870 Mr. Frizell opened an office for general consulting practice in Boston, where he remained until 1878. He made many hydraulic investigations, including reports on the Minneapolis water-power in 1875, and on damages from diversions on the Charles River, Waltham, and on the Neponset River and Mother Brook. In 1878 he went West as Assistant Civil Engineer to the U. S. Engineering Department, and was engaged in various hydraulic investigations, among which were the construction of reservoirs at the head-waters of the Mississippi River, investigations at the Falls of St. Anthony, and other similar matters. In 1881 he served on the Water Commission at St. Paul,

* Memoir prepared by R. A. Hale, M. Am. Soc. C. E.

reporting on the water supply. During his work throughout the West, in the U. S. Engineering Department, the letters received by him from various army officers showed the high esteem in which he was held.

In 1890-92 Mr. Frizell was Chief Engineer of the Board of Public Works at Austin, Tex., during which time he made a design for, and commenced the construction of, the Austin Dam. On account of various restrictions by the Board of Public Works, he resigned this position in June, 1892, after which the dam was completed upon a somewhat modified plan. The general history of this work is stated in various engineering publications.

In 1893 he resumed his Boston office and practised until 1903, when he retired from active business. During this last period in Boston Mr. Frizell was connected with various water-diversion cases on the Nashua River due to the taking by the Metropolitan Water Board.

In 1878 he obtained patents for compressing air by hydraulic methods, using inverted siphons and air chambers, etc., and also for pumping water by the use of compressed air, both of which have found practical application. The first method is described in detail in a paper by Mr. William Webber, entitled "Water Power by Direct Air Compression."* Air compressor plants built on these general principles have been installed at Norwich, Conn., Magog, Canada, and are in use in European countries. The Pohle air-lift pump is an illustration of the application of other methods.

In 1900 Mr. Frizell published a book on "Water Power" which was an interesting addition to hydraulic literature, and he was also a frequent contributor to various scientific publications. Among them is an article on the "Strains in Continuous Beams."† He was collaborator on Johnson's Civil Engineering Cyclopedia. Among the papers contributed to the *Transactions* of the American Society of Civil Engineers may be mentioned "The Water-Power of the Falls of St. Anthony," Vol. XII; "The Storage and Pondage of Water," Vol. XXXI; and "The Old Time Water Wheels of America," Vol. XXVIII. He was interested in turbine water-wheels, and made a study in developing a formula by which the discharge of a turbine of the Boyden type could be computed from the various angles of the guides and buckets. He was a remarkable mathematician, and was self-taught in the higher mathematics.

His son, Dr. Arthur B. Frizell, writes as follows:

"My father's professional reputation was essentially scientific; he was valued for his ability to carry through a piece of work requiring original investigation. His success in dealing with difficult problems was owing to mathematical aptitudes nearly akin to genius.

* *Journal*, Association of Engineering Societies, Vol. 26, p. 35.

† *Journal*, Franklin Institute, 1872.

With the faculty of formulating engineering questions mathematically he combined rare skill in the use of the delicate analysis by aid of infinitesimals. To-day such faculty is generally the result of protracted academic training in methods more accessible to the average mind; but my father was self-taught, at least in calculus; his processes were intuitive and sure.

"There is an interesting entry in his diary to the effect that the ideas on which calculus is based admit of much wider application, indeed, are capable of guiding us in every study that proceeds by exact reasoning. Fifty years later, Professor Schoenflies of Königsberg, in a report on the progress on the newest branch of mathematics, which is largely concerned with precisely these underlying principles of calculus, asserted that its development from a set of axioms, analogous to those of Euclid, would be equivalent to an analysis of the foundations of all human knowledge."

From 1903 until his death in 1910, Mr. Frizell resided in Dorchester, Mass. He was a member of the Tariff Reform League, and took an active interest in writing papers on the subject. He was a member of the Unitarian Church, which he attended regularly with his family. Although of a quiet and somewhat reserved disposition, he had many warm friends, and the personal letters of Hon. George F. Hoar, George S. Shattuck, Esq., and other prominent men, testify to their appreciation of his ability and friendship.

On October 17th, 1864, Mr. Frizell was married to Julia A. Bowes, of Boston, Mass. He is survived by Mrs. Frizell and two children, Dr. Arthur B. Frizell, a member of the faculty of Kansas State University, and Miss Alice M. Frizell, who resides with her mother in Cambridge, Mass.

Mr. Frizell was elected a Member of the American Society of Civil Engineers on January 3d, 1883.

JOHN FRANKLIN HINCKLEY, M. Am. Soc. C. E.*

DIED FEBRUARY 20TH, 1911.

John Franklin Hinckley was born in Boston, Mass., on December 13th, 1849. His parents were John Loving Hinckley and Emeline Hinckley. He received his education in the public schools and at the English High School of Boston.

His father was engaged in the railroad business in the West, and in July, 1868, Mr. Hinckley joined the Engineering Corps of the Atlantic and Pacific Railroad (now the St. Louis and San Francisco Railroad), and was Assistant Engineer on its construction from April, 1869, until May, 1870, and Assistant Engineer on surveys until July, 1871, at which time he entered the service of the Missouri Pacific and was employed on the construction of its Lexington Branch.

From January 1st, 1872, to August, 1873, Mr. Hinckley was Assistant Engineer on the Indiana and Illinois Central Railroad, and from August, 1873, to December, 1875, he was employed as Chief Engineer on the Havana, Rantoul and Eastern Railroad. In December, 1878, he was appointed Division Engineer in charge of the construction of 50 miles of the Chillicothe, Brunswick and Omaha Railroad. In July, 1879, he entered the service of the St. Louis and San Francisco Railroad, remaining in the Company's employ as Division Engineer and Principal Assistant Engineer until January, 1887, when he was appointed Chief Engineer of the St. Louis, Arkansas and Texas Railway (now the St. Louis Southwestern Railway), and had charge of the construction of branches to Shreveport, La., Hillsboro, Tex., Little Rock, Ark., Fort Worth and Sherman, Tex., until July, 1888, when he resigned.

In 1888 and 1889, Mr. Hinckley was Assistant to Robert Moore, Past-President, Am. Soc. C. E., on the Merchants Bridge Terminal Railway in St. Louis, Mo., and from 1889 to 1894, he was associated with Messrs. A. W. Scott and J. H. Hedges in contracting work and in a quarry. During this time he did a great deal of work for the Kansas City, Nevada and Fort Smith Railroad and for the Missouri, Kansas and Texas Railway, and was also engaged in private practice.

In August, 1894, Mr. Hinckley was appointed Chief Engineer of the Choctaw, Oklahoma and Gulf Railroad (now the Rock Island Company), and constructed the line from McAllister to Oklahoma City. In January, 1896, he was engaged as Chief Engineer of Construction of the St. Louis and San Francisco Railroad, and had charge of the construction of many miles of this road, mostly in Oklahoma and Texas, including the lines from Bolivar to Osceola, Mo.; from Sapulpa

*Memoir prepared by C. D. Purdon and F. G. Jonah, Members, Am. Soc. C. E.

to Oklahoma City, Okla.; from Sapulpa, Okla., to Sherman, Tex.; from Tulsa to Avard, Okla.; and from Sherman to Carrolton, Tex. On the Fort Worth and Rio Grande Railway, he was in charge of the construction from Brownwood to Brady, Tex., and supervised the construction of the line from Hope to Ardmore, Mo., and from Oklahoma City to Quanah, Tex. He also supervised the construction of the Blackwell, Enid and Southwestern Railway, from Okeen, Okla., to Vernon, Tex.

In January, 1904, Mr. Hinckley was appointed Chief Engineer of the Frisco, remaining in this position until February, 1909. During this time he supervised the construction of the St. Louis, Brownville and Mexico Railway, the Colorado Southern, the New Orleans and Pacific, and the New Orleans Terminal Railway. During his service with this road (1896-1909), about 1 800 miles of the present "Frisco System" were built under his supervision. On his resignation as Chief Engineer, in February, 1909, he engaged in private practice, opening an office in St. Louis, Mo. His death, which occurred on February 20th, 1911, after a short illness, was caused by pneumonia.

Mr. Hinckley was closely identified with the development of what is now the State of Oklahoma through his work of railroad construction. He was a courteous man, kind and considerate to his subordinates and ever willing to promote the younger men in his employ; consequently, he had the esteem and confidence of his assistants to a marked degree. He was as studious as permitted by his active life and owned one of the best libraries in St. Louis. Although he did not seek general society, he was hospitable and friendly, especially to those of his Profession, and he will be greatly missed by all who knew him.

Mr. Hinckley was elected a Member of the American Society of Civil Engineers on May 6th, 1885. He was also a Member of the American Railway Engineering and Maintenance of Way Association, the American Society for Testing Materials, the Engineers' Club of St. Louis, and other societies.

HENRY RANDOLPH HOLBROOK, M. Am. Soc. C. E.*

DIED DECEMBER 21ST, 1907.

Henry Randolph Holbrook, son of Nathan K. and Jane (Porter) Holbrook, was born on November 22d, 1838, at Columbia, Conn. He was educated in the common schools, and at the age of 17 left home to make his way in the world.

From February, 1865, to September, 1866, Mr. Holbrook was engaged as Assistant Engineer in charge of construction on the Missouri Pacific Railroad. In October, 1866, he joined the Engineer Corps of the Eastern Division of the Union Pacific Railroad, remaining with the Corps until August, 1868.

In April, 1867, an Engineer Corps, in charge of the late William H. Greenwood, M. Am. Soc. C. E., as Chief Engineer, Mr. Holbrook being his Principal Assistant Engineer, left Junction City, Kans., to make a survey of the line to Denver, Colo. After locating the line between Forts Hays and Elsworth and continuing the preliminary surveys to Denver, the Union Pacific Railroad Company decided to make a survey through to San Diego, Cal. Five parties were sent out, two on the 32d Parallel and three on the 35th Parallel, Mr. Holbrook having charge of one of the latter parties.

From August, 1868, to September, 1869, he had charge of the location of the St. Louis and San Francisco Railroad from Jerome to Springfield, Mo., a distance of 100 miles, and from September, 1869, to September, 1870, he was employed in locating and had charge of the construction of 70 miles on the Kansas Branch of the Union Pacific Railroad, from Kit Carson, Colo., westward.

Mr. Holbrook was Chief Engineer of the Denver and Boulder Valley Railroad during its construction, from September, 1870, to February, 1871, after which he was in charge of a locating party on the Denver and Rio Grande Railroad. In March, 1872, he went to Mexico, where he remained until July, making reconnaissances for railroads in connection with this same road.

From July, 1874, to December, 1875, he was employed as Chief Engineer on the construction of the Pueblo and Salt Lake Railroad, from Las Animas to Pueblo, Colo., and was Resident Engineer on the Atchison, Topeka, and Santa Fé Railroad from March, 1878, to July, 1880.

The Eastern Division of the Union Pacific Railroad was not built beyond Denver, Colo., the Southern Pacific ultimately using the 32d Parallel for its route and the Atchison, Topeka, and Santa Fé the

*Memoir prepared by Augustine W. Wright, M. Am. Soc. C. E.

35th Parallel, and from July, 1880, to January, 1882, Mr. Holbrook was Chief Engineer of the Atlantic and Pacific portions of the latter line.

In 1882, he returned to Mexico, where he spent that year and 1883 in making examinations for proposed railroads.

In April, 1887, Mr. Holbrook was again appointed Chief Engineer of the Pueblo and Salt Lake Railroad, remaining in that position until 1890, from which time until his death, on December 21st, 1907, he devoted himself to irrigation and real estate projects, making his home at Pueblo, Colo., where he died. A brother and three sisters survive him.

Mr. Holbrook will always be remembered for "the best portion of a good man's life: his little, nameless, unremembered acts of kindness and of love."

He was elected a Member of the American Society of Civil Engineers on January 2d, 1890.

LOUIS EDWIN HAWES, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 29TH, 1911.

Louis Edwin Hawes, son of Henry E. and Frances E. (Wesson) Hawes, was born in Springfield, Mass., on January 27th, 1860, his mother being a sister of the late Daniel B. Wesson, head of the world-famous firm manufacturing the Smith and Wesson revolver.

His youth was passed in his native city, where, at the age of fifteen, as the eldest child of a widowed mother with six children, the stern duty of caring for and maintaining the family was suddenly thrust upon him. That this duty was faithfully performed is attested by the filial affection and deep family devotion which characterized Mr. Hawes throughout his life.

He studied at the Worcester, Mass., Polytechnic Institute, where he entered with zest into the sports of the day, being especially skilled in base-ball. He played the violin in the orchestra of the Institute. On this graduation, in 1882, with the degree of B. S., he at once began active work in his chosen profession. In the summer of 1882 he was Levelman and Assistant Engineer on the location survey for the Meriden and Cromwell Railroad in Connecticut, and later in the year, Assistant Engineer on water-works construction at Northboro, Mass. During 1883 and 1884, he was Assistant Engineer on the Wakefield, Stoneham, and North Attleboro, Mass., water-works. In 1885 he was Resident Engineer on the Norwood, Mass., water-works, and in 1886 served in the same capacity on the Juniper Hill Reservoir, at Rockland, Me., and was in charge of the surveys for a sewerage system at Hyde Park, Mass.

In 1887 he was employed as Resident Engineer on the construction of the Ayer, Mass., water-works, and, in 1888, as Assistant Engineer on the Dover, N. H., water-works, his work for the preceding five years having been under Mr. Percy M. Blake.

In 1889, Mr. Hawes began business for himself, opening an office in Boston, Mass. He continued in active practice as a civil and hydraulic engineer and as a contractor until his death, a period of twenty-two years. During this time he investigated, designed, and reported on or constructed new water-works systems or improved old ones in about forty towns and cities in New England and the West, and was frequently called as an expert in the appraisal of water-supply plants, in legal controversies, and in other matters involving hydraulic questions. Among the places where he did work may be mentioned, Avon, Needham, Provincetown, Lexington, Holden, Housatonic, Falmouth, Amesbury, Milford, Whitman, Edgartown, and Marion, in Massachusetts;

* Memoir prepared by George M. Warren, Esq.

Middlebury, Vt.; Alton, Ill., and Independence and Newton, Kans. He designed and reported on a system of sewerage and sewage disposal for Wakefield, Mass., and, in 1895, was Chairman of the Sewerage Commission of that town. At the time of his death he was Treasurer and Manager of the Edgartown Water Company.

Mr. Hawes was a man of pleasing personality, unswerving integrity, strong religious conviction, and, as an engineer, thoroughly reliable, painstaking, and observant of every detail. During a close professional association of more than ten years, the writer never heard him utter a violent or unchaste remark or speak ill of any one. One of his contemporaries writes:

"Mr. Hawes was a careful and painstaking engineer of ability and good judgment. Although modest and retiring in his manner, he gave to all matters entrusted to his attention a full consideration and the benefit of a well-balanced judgment. Whatever he did, he did well."

The Chairman of the Water Commissioners of Holden, Mass., referring to the monthly estimates prepared by Mr. Hawes, wrote: "They are models of clearness and mathematical accuracy."

On June 16th, 1886, Mr. Hawes married Miss Hattie M. Emerson, of Wakefield, Mass., who, with a daughter and a son, survives him.

Mr. Hawes was admitted to the New England Water-Works Association, on December 12th, 1888; and to the Boston Society of Civil Engineers, on June 20th, 1894.

He was elected an Associate Member of the American Society of Civil Engineers, on September 2d, 1896.

ALBERT VICTOR KELLOGG, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 15TH, 1911.

Albert Victor Kellogg was born at Aurora, Ill., on August 4th, 1863. He was the third son of the late Professor J. F. Kellogg, who occupied the head chair of Mathematics at Northwestern University, Evanston, Ill., where his son received his collegiate training.

Mr. Kellogg had given special attention to mathematics, and, having natural inclination and talent for engineering work, he began his career, in 1881, as a Rodman on location in the Engineering Department of the Chicago and Northwestern Railway. In 1884 he became Assistant Engineer of the Chicago and Evanston Railway, a part of the Chicago, Minneapolis, and St. Paul, and later was employed on surveys for the location of the Fremont, Elkhorn, and Missouri Valley Railway (now a part of the Chicago and Northwestern Lines), on which he worked as Assistant Engineer.

During the interim between this and subsequent railway work, Mr. Kellogg was engaged in municipal engineering in Omaha and neighboring cities in Nebraska, being associated with the late Andrew Rosewater, M. Am. Soc. C. E. He was employed principally as Assistant Engineer in charge of the surveys and construction of the sewerage systems of Grand Island, Hastings, and Plattsmouth, Nebr. Following this work, he was engaged in the general practice of engineering in St. Joseph, Mo., and also in Denison and other cities in Texas.

About 1890, he re-entered railway service, and was connected with several Texas lines, notably the Missouri, Kansas, and Texas Railway, on which he held a responsible position in charge of construction and later in the Engineering Department of the operated line.

In 1893, when construction work was hindered by the financial panic, Mr. Kellogg became Engineer of the Right-of-Way Department of the Houston and Texas Central Railroad, at Houston, Tex., where he rendered valuable service in re-surveys, and in perfecting and systematizing records. He established himself so firmly in the confidence of the officers that he was put in charge of the next construction work undertaken by that Company. This work was done so thoroughly and so well that his ability was recognized, and, as a result, in 1901, he was put in charge of the Maintenance-of-Way Department of the Houston and Texas Central, and Houston East and West Texas Lines, subsidiary lines to the Southern Pacific Railway, Atlantic System. He continued in this position until 1904, when he was appointed Engineer of Maintenance of Way of the Sunset Lines, the Atlantic System of the Southern Pacific Railway, comprising the Morgan Louisiana and

* Memoir prepared by R. D. Parker, Assoc. M. Am. Soc. C. E.

Texas Railroad and Steamship Company, the Louisiana Western Railroad Company, Texas and New Orleans Railroad Company, and the Galveston, Harrisburg, and San Antonio Railway Company, with headquarters at Houston, Tex., which position he held until his death at Santa Rosa Hospital in San Antonio, Tex., on February 15th, 1911.

Mr. Kellogg possessed the qualities which go into the make-up of a big man. We who are left but the memory of this friend, appreciate the example he gave us in his energetic work, his fidelity to a trust and to a friend, and his loyalty to a service set apart for him. His life was that of a builder for the benefit of mankind, and his work was well done.

Mr. Kellogg was elected an Associate Member of the American Society of Civil Engineers on November 6th, 1895.

TRANSACTIONS

OF THE

American Society of Civil Engineers

INDEX

VOLUME LXXIII

SEPTEMBER, 1911

SUBJECT INDEX, PAGE 514

AUTHOR INDEX, PAGE 517

Titles of papers are in quotation marks when given with the author's name.

VOLUME LXXIII

SUBJECT INDEX

ADDRESSES.

"Address at the 43d Annual Convention, Chattanooga, Tennessee, June 13th, 1911." Mordecai T. Endicott. 392.

BEAMS.

"Bond-Friction-Resistance in Reinforced Concrete." William Fry Scott. (With Discussion.) 230.

BRIDGES.

"Some Tests of Large Steel Columns." James E. Howard. (With Discussion.) 429.

"The Pittsburg and Lake Erie Railroad Cantilever Bridge Over the Ohio River at Beaver, Pa." Albert R. Raymer. 136. Discussion: C. W. Hudson, Henry S. Prichard, Albert B. Hager, F. W. Skinner, A. W. Carpenter, and Albert Lucius, 166.

CANTILEVER BRIDGES.

"The Pittsburg and Lake Erie Railroad Cantilever Bridge Over the Ohio River at Beaver, Pa." Albert R. Raymer. (With Discussion.) 136.

COLUMNS.

"Some Tests of Large Steel —." James E. Howard. 429. Discussion: Albert Lucius, Leon S. Moisseiff, A. W. Carpenter, J. S. Branne, Lewis D. Rights, J. R. Worcester, Horace E. Horton, Arthur N. Talbot, James Christie, N. R. McLure, R. S. Chew, George N. Cole, F. P. Shearwood, Charles Worthington, and F. C. Kunz, 448.

DAMS.

"— on Sand Foundations: Some Principles Involved in Their Design, and the Law Governing the Depth of Penetration Required for Sheet-Piling." Arnold C. Koenig. 175. Discussion: W. G. Price, Alexander Potter, T. Kennard Thomson, G. E. P. Smith, Allen Hazen, and R. C. Beardsley, 190.

FILTRATION.

The flow of underground water through sand, gravel, etc., as in seepage or percolation. 176.

FINANCE.

"The Going Value of Water-Works." Leonard Metcalf and John W. Alvord. (With Discussion.) 326.

FOUNDATIONS.

"Dams on Sand —: Some Principles Involved in Their Design, and the Law Governing the Depth of Penetration Required for Sheet-Piling." Arnold C. Koenig. (With Discussion.) 175.

GROUTING.

The use of grout for stopping leaks in shafts and tunnels. 404.

MEMOIRS OF DECEASED MEMBERS.

Buck, Leffert Lefferts. 493.
Caldwell, George Bowers. 498.
Davis, Frank Leslie. 500.
Frizell, Joseph Palmer. 501.
Hawes, Louis Edwin. 508.
Hinckley, John Franklin. 504.
Holbrook, Henry Randolph. 506.
Kellogg, Albert Victor. 510.

PAVEMENT.

"Road Construction and Maintenance." An Informal Discussion. Logan Waller Page and others. 1.

"Street Paving Crowns, Washington, D. C." T. J. Powell. 225. Discussion: E. A. Steece, 228.

PILES.

"Dams on Sand Foundations: Some Principles Involved in Their Design, and the Law Governing the Depth of Penetration Required for Sheet-Piling." Arnold C. Koenig. (With Discussion.) 175.

REINFORCED CONCRETE.

"Bond-Friction-Resistance in —." William Fry Scott. 230. Discussion: Edward Godfrey, Joseph T. Maguire, A. N. Talbot, Hector Robins Burroughs, Albert I. Frye, and F. E. Turneaure, 267.

ROADS.

"Road Construction and Maintenance." An Informal Discussion. Logan Waller Page, W. W. Crosby, Arthur H. Blanchard, Clifford Richardson, C. F. Knowlton, A. F. Armstrong, Watson G. Clark, Harold Parker, Henry B. Drowne, Joseph W. Hunter, Paul D. Sargent, Fred. E. Ellis, Samuel Whinery, H. B. Pullar, Nelson P. Lewis, Hubert K. Bishop, George C. Diehl, R. A. Meeker, James H. MacDonald, J. L. Wickes, H. C. Poore, Charles W. Ross, L. R. Grabill, Prevost Hubbard, Frank J. Eppele, John R. Rablin, R. K. Compton, William H. Connell, Arthur W. Dean, Michael Driscoll, Philip P. Sharples, James Owen, Malcolm H. Smith, E. H. Rogers, H. G. Shirley, James C. Wonders, F. C. Pillsbury, F. P. Smith, and J. W. Howard. 1.

Use of water, calcium chloride, light oils, etc., as dust palliatives. 33.

SHAFT SINKING.

"Sinking a Wet Shaft." John P. Hogan. 398. Discussion: Mason D. Pratt, Charles B. Buerger, L. White, H. M. Hale, H. F. Dunham, T. Kennard Thomson, Bertrand H. Wait, T. H. Wiggin, and V. H. Hewes, 414.

STEEL.

"Some Tests of Large — Columns." James E. Howard. (With Discussion.) 429.

STRENGTH OF MATERIALS.

"Bond-Friction-Resistance in Reinforced Concrete." William Fry Scott. (With Discussion.) 230.

TRAFFIC.

Methods of taking — census. 1.

TUNNELS.

"Sinking a Wet Shaft." John P. Hogan. (With Discussion.) 398.

VALUATION.

"The Going Value of Water-Works." Leonard Metcalf and John W. Alvord. (With Discussion.) 326.

WATER-WORKS.

"The Going Value of —." Leonard Metcalf and John W. Alvord. 326. Discussion: Clinton S. Burns, Albert I. Frye, Frederic P. Stearns, E. Kuichling, H. F. Dunham, Charles B. Buerger, Halbert P. Gillette, and George W. Fuller, 356.

AUTHOR INDEX

ALVORD, JOHN W.
"The Going Value of Water-Works." 326.

ARMSTRONG, A. F.
Road construction and maintenance. 10, 89.

BEARDSLEY, R. C.
Dams on sand foundations. 203.

BISHOP, HUBERT K.
Road construction and maintenance. 25, 41, 70.

BLANCHARD, ARTHUR H.
Road construction and maintenance. 7, 56, 99, 134.

BRANNE, J. S.
Tests of large steel columns. 457.

BUCK, LEFFERT LEFFERTS.
Memoir of. 493.

BUERGER, CHARLES B.
Going value of water-works. 375.
Sinking a wet shaft. 415.

BURNS, CLINTON S.
Going value of water-works. 356.

BURROUGHS, HECTOR ROBINS.
Bond-friction-resistance in reinforced concrete. 276.

CALDWELL, GEORGE BOWERS.
Memoir of. 498.

CARPENTER, A. W.
Railroad cantilever bridge at Beaver, Pa. 172.
Tests of large steel columns. 452.

CHEW, R. S.
Tests of large steel columns. 469.

CHRISTIE, JAMES.
Tests of large steel columns. 467.

CLARK, WATSON G.
Road construction and maintenance. 12.

COLE, GEORGE N.
Tests of large steel columns. 470.

COMPTON, R. K.
Road construction and maintenance. 63, 78.

CONNELL, WILLIAM H.
Road construction and maintenance. 64, 81, 109.

CROSBY, W. W.
Road construction and maintenance. 4, 19, 49, 74, 96.

DAVIS, FRANK LESLIE.
Memoir of. 500.

DEAN, ARTHUR W.
Road construction and maintenance. 65.

DIEHL, GEORGE C.
Road construction and maintenance. 28.

DRISCOLL, MICHAEL.
Road construction and maintenance. 68, 88.

DROWNE, HENRY B.
Road construction and maintenance. 15.

DUNHAM, H. F.
Going value of water-works. 373.
Sinking a wet shaft. 418.

ELLIS, FRED. E
Road construction and maintenance. 20, 67.

ENDICOTT, MORDECAI T.
"Address at the 43d Annual Convention, Chattanooga, Tennessee, June 13th, 1911." 392.

EPPELE, FRANK J.
Road construction and maintenance. 57, 108.

FRIZELL, JOSEPH PALMER.
Memoir of. 501.

FRYE, ALBERT I.
Bond-friction-resistance in reinforced concrete. 278.
Going value of water-works. 361.

FULLER, GEORGE W.
Going value of water-works. 384.

GILLETTE, HALBERT P.
Going value of water-works. 378.

GODFREY, EDWARD.
Bond-friction-resistance in reinforced concrete. 267.

GRABILL, L. R.
Road construction and maintenance. 42.

HAGER, ALBERT B.
Railroad cantilever bridge at Beaver, Pa. 169.

HALE, H. M.
Sinking a wet shaft. 417.

HAWES, LOUIS EDWIN.
Memoir of. 508.

HAZEN, ALLEN.
Dams on sand foundations. 199.

HEWES, V. H.
Sinking a wet shaft. 423.

HINCKLEY, JOHN FRANKLIN.
Memoir of. 504.

HOGAN, JOHN P.
"Sinking a Wet Shaft." 398.

HOLBROOK, HENRY RANDOLPH.
Memoir of. 506.

HORTON, HORACE E.
Tests of large steel columns. 461.

HOWARD, J. W.
Road construction and maintenance. 132.

HOWARD, JAMES E.
"Some Tests of Large Steel Columns." 429.

HUBBARD, PREVOST.
Road construction and maintenance. 54.

HUDSON, C. W.
Railroad cantilever bridge at Beaver, Pa. 166.

HUNTER, JOSEPH W.
Road construction and maintenance. 19.

KELLOGG, ALBERT VICTOR.
Memoir of. 510.

KNOWLTON, C. F.
Road construction and maintenance. 10.

KOENIG, ARNOLD C.
"Dams on Sand Foundations: Some Principles Involved in Their Design, and the Law Governing the Depth of Penetration Required for Sheet-Piling." 175.

KUICHLING, E.
Going value of water-works. 368.

KUNZ, F. C.
Tests of large steel columns. 484.

LEWIS, NELSON P.
Road construction and maintenance. 23.

LUCIUS, ALBERT.
Railroad cantilever bridge at Beaver, Pa. 173.
Tests of large steel columns. 448.

MACDONALD, JAMES H.
Road construction and maintenance. 31.

McLURE, N. R.
Tests of large steel columns. 467.

MAGUIRE, JOSEPH T.
Bond-friction-resistance in reinforced concrete. 271.

MEEKER, R. A.
Road construction and maintenance. 30, 42, 71, 94, 128.

METCALF, LEONARD.
"The Going Value of Water-Works." 326.

MOISSEIFF, LEON S.
Tests of large steel columns. 449.

OWEN, JAMES.
Road construction and maintenance. 71.

PAGE, LOGAN WALLER.
Road construction and maintenance. 2,-31.

PARKER, HAROLD.
Road construction and maintenance. 13, 24, 67.

PILLSBURY, F. C.
Road construction and maintenance. 106.

POORE, H. C.
Road construction and maintenance. 40.

POTTER, ALEXANDER.
Dams on sand foundations. 191.

POWELL, T. J.
"Street Paving Crowns, Washington, D. C." 225.

PRATT, MASON D.
Sinking a wet shaft. 414.

PRICE, W. G.
Dams on sand foundations. 190.

PRICHARD, HENRY S.
Railroad cantilever bridge at Beaver, Pa. 167.

PULLAR, H. B.
Road construction and maintenance. 22.

RABLIN, JOHN R.
Road construction and maintenance. 61.

RAYMER, ALBERT R.
"The Pittsburg and Lake Erie Railroad Cantilever Bridge Over the Ohio River at Beaver, Pa." 136.

RICHARDSON, CLIFFORD.
Road construction and maintenance. 9, 131.

RIGHTS, LEWIS D.
Tests of large steel columns. 458.

ROGERS, E. H.
Road construction and maintenance. 89.

ROSS, CHARLES W.
Road construction and maintenance. 41, 44, 73, 77.

SARGENT, PAUL D.
Road construction and maintenance, 20, 93, 132.

SCOTT, WILLIAM FRY.
"Bond-Friction-Resistance in Reinforced Concrete." 230.

SHARPLES, PHILIP P.
Road construction and maintenance. 69.

SHEARWOOD, F. P.
Tests of large steel columns. 471.

SHIRLEY, H. G.
Road construction and maintenance. 92.

SKINNER, F. W.
Railroad cantilever bridge at Beaver, Pa. 171.

SMITH, F. P.
Road construction and maintenance. 128.

SMITH, G. E. P.
Dams on sand foundations. 193.

SMITH, MALCOLM H.
Road construction and maintenance. 89.

STEARNS, FREDERIC P.
Going value of water-works. 362.

STEECE, E. A.
Street paving crowns. 228.

TALBOT, A. N.
Bond-friction-resistance in reinforced concrete. 274.
Tests of large steel columns. 465.

THOMSON, T. KENNARD.
Dams on sand foundations. 193.
Sinking a wet shaft. 419.

TURNEAURE, F. E.
Bond-friction-resistance in reinforced concrete. 286.

WAIT, BERTRAND H.
Sinking a wet shaft. 420.

WHITE, L.
Sinking a wet shaft. 415.

WICKES, J. L.
Road construction and maintenance. 37.

WHINERY, SAMUEL.
Road construction and maintenance. 21, 33, 42, 95.

WIGGIN, T. H.
Sinking a wet shaft. 420.

WONDERS, JAMES C.
Road construction and maintenance. 95.

WORCESTER, J. R.
Tests of large steel columns. 460.

WORTHINGTON, CHARLES.
Tests of large steel columns. 471.

TA
1
A5
V.73